PART II PERMIT APPLICATION
SCEPTER, INC. DISPOSAL FACILITY – EAST PHASE SITE
ENGINEERING PLANS AND NARRATIVES

Prepared for
Scepter, Inc
Waverly, TN

July 1, 2016

December 6, 2017 Revised (all)
April 5, 2018 Rev

Prepared by

AECOM

TABLE OF CONTENTS

REPORT CERTIFICATION

I certify under penalty of law that this document was prepared by me or under my supervision in accordance with a system designed to assure that qualified personnel properly gathered and evaluated the information submitted. The information submitted is, to the best of my knowledge and belief, true, accurate, and complete.

MICHAEL S. MAY
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
NO. 103104
STATE OF TENNESSEE

Michael May
4-5-18

Michael S. May, P.E.
July 1, 2016
April 5, 2018 (Rev)

Adopted From:
TDEC Division of Solid Waste Management
SOP for Solid Waste Disposal Facility Permit
Revision #: 4
Effective Date: September, 2006

"Completeness Review" Checklist

Part A:		Operations Manual	TDEC Reference Chapter 0400-11-01, Paragraph	Reference Part A
1.	Owner Responsible Person	Scepter, Inc. General Manager	.04(9)(c)1. .04(9)(c)1.	Section 2.1
2.	Location of facility Address	Humphreys County 1485 Scepter Lane Waverly, Tennessee 37185	.04(9)(c)2. .04(9)(c)2.	Section 2.2
3.	Compliance with buffer zone(s)		.04(3)(b) .04(9)(c)3.	Section 2.3
4.	The facility is not located within 200' of a fault area that had displacement		.04(2)(u) .04(9)(c)4.	Section 2.4
5.	The facility is not located in seismic impact area or zone		.04(2)(v) .04(9)(c)5.	Section 2.5
6.	In unstable area the operator must consider the following:			
	(i)	On site or local soil conditions for differential settlement	.04(2)(w)1. .04(9)(c)6.	Section 2.6
	(ii)	On site or local geologic or geomorphologic features	.04(2)(w)2. .04(9)(c)6.	Section 2.6
	(iii)	On site or local human-made features (both surface or subsurface)	.04(2)(w)3. .04(9)(c)6.	Section 2.6
7.	Access to and use of facility		.04(2)(b) .04(9)(c)7.	Section 2.7
8.	Methods and sequence of operation		.04(9)(c)8.	Section 2.8

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Part A:	Operations Manual (continued)		TDEC Reference Chapter 0400-11-01, Paragraph	Reference Part A
9.	Types and anticipated volumes of waste: (check appropriate type and indicate amount)		.04(9)(c)9.	Section 2.9
10.	Acres to be filled and acres permitted		.04(9)(c)10.	Section 2.10
11.	Waste handling and covering program			
	(i)	Unloading, spreading, compacting	.04(6)(b)1. .04(9)(c)11.(i)	Section 2.11
	(ii)	Frequencies and depths of cover (3 types)	.04(6)(b)2.(i) .04(9)(c)11.(ii)	Section 2.11
	(iii)	Soil balance/availability of cover	.04(6)(b)2.(i) .04(9)(c)11.(iii)	Section 2.11
12.	Operating equipment		.04(2)(g) .04(9)(c)12.	Section 2.12
13.	Procedure for controlling and collecting blowing litter		.04(2)(d) .04(9)(c)13.	Section 2.13
14.	Management of erosion control facilities		.04(2)(i)6. .04(9)(c)14.	Section 2.14
15.	Management of leachate collection facilities		.04(4)(b) .04(9)(c)15.	Section 2.15
16.	Dust control measures and implementation		.04(2)(j) .04(9)(c)16.	Section 2.16
17.	Fire safety precautions		.04(2)(c) .04(9)(c)17.	Section 2.17
18.	Facility services		.04(2)(e) .04(9)(c)18.	Section 2.18

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Part A:		Operations Manual (continued)	TDEC Reference Chapter 0400-11-01, Paragraph	Reference Part A
19.	Inspection of liners and cover systems			
	(i)	Any new phase or expansion should be tested and inspected by a P.E.	.04(9)(c)19.(i)	Section 2.19 & Part D CQA Plan
	(ii)	Each section should be certified by a P.E.	.04(9)(c)19.(ii)	Section 2.19 & Part D CQA Plan
20.	Containment of migration of explosive gases		.04(5)(b) .04(9)(c)20.	Section 2.20
21.	Planned GW monitoring program			
	(i)	Number and location of wells	.04(7)(a)3.(i) .04(9)(c)21.(i)	Section 2.21
	(ii)	Monitoring well construction	.04(7)(a)3.(ii) .04(9)(c)21.(ii)	Section 2.21
	(iii)	Parameters to be monitored	.04(7)(b)2. .04(7)(b)6. .04(9)(c)21.(iii)	Section 2.21
	(iv)	Sampling and procedures	.04(7)(b)3. .04(9)(c)21.(iv)	Section 2.21
	(v)	How sampling and results will be recorded and reported	.04(7)(b)4. .04(7)(b)5. .04(9)(c)21.(v)	Section 2.21
22.	Engineering statement of site flood frequency		.04(9)(c)22.	Section 2.22
23.	Impacts on endangered or threatened species		.04(2)(m) .04(9)(c)23.	Section 2.23
24.	Random inspection program		.04(2)(s) .04(9)(c)24.	Section 2.24

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Part B:		Closure/Post Closure Care Plan	TDEC Reference Chapter 0400-11-01, Paragraph	Reference Part B
1.	Contents of Plan			
	(i)	Plan identifies steps necessary to completely or partially close the facility at any point during intended operating life	.03(2)(b)1.	Section 2.1
	(ii)	Identifies steps to completely close at end of intended operating life	.03(2)(b)1. .04(8)(c)	Section 2.1
	(iii)	Identifies activities after closure and frequency of activities	.03(2)(b)1. .04(8)(d) .04(8)(e)	Section 3.2
	(iv)	For phased development facilities, plan addresses each parcel separately as well as the whole	.03(2)(b)1.	Section 2.1
2.	Plan includes a description of:			
	(i)	How and when facility will be partially and finally closed. Also includes expected year of closure	.03(2)(b)2.(i)	Sections 1.2 & 2.1
	(ii)	Planned Groundwater and Surface water monitoring and maintenance activities and frequencies	.03(2)(b)2.(ii)	Section 3.2.5 & Part C GWM Plan
	(iii)	Person or office, name and number to contact during post closure	.03(2)(b)2.(iii)	Section 1.3
	(iv)	Itemized estimate of third party cost of performing closure and post closure	.03(2)(b)2.(iv)	Appendix B.1
	(v)	Planned uses of property during post-closure period	.03(2)(b)2.(v)	Section 3.3
3.	In closure plan, operation addresses closure of active portions and future active portions of facility. In post-closure care, operator addresses post-closure care of closed, active, and future active portions		.03(2)(b)3.	Sections 2.0 & 3.0

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Part E:		Engineering Plans	TDEC Reference Chapter 0400-11-01, Paragraph	Reference Part E Drawing P-
1.	Plans, drawn at a scale not less than 1" = 100' and contour interval no more than 5' that show locations of and/or describe:			
	(i)	Proposed waste disposal areas	.04(9)(b)1.(i)	3
	(ii)	Existing topography with pertinent features	.04(9)(b)1.(ii)	2
	(iii)	On-site benchmarks, with reference to TN datum	.04(9)(b)1.(iii)	3
	(iv)	GW, SW monitoring points and compliance boundary	.04(9)(b)1.(iv)	4
	(v)	Soil boring locations	.04(9)(b)1.(v)	4
	(vi)	Dikes, berms, trenches, excavation contours	.04(9)(b)1.(vi)	4
	(vii)	Borrow and cover material storage area	.04(9)(b)1.(vii)	5
	(viii)	Planned development of site (phases)	.04(9)(b)1.(viii)	10,11,12, 13
	(ix)	Temporary/permanent erosion control measures	.04(9)(b)1.(x)	10 through 14
	(x)	Run-on/run-off diversions from work areas and facility	.04(9)(b)1.(ix)	14
	(xi)	Existing/proposed utilities, structures, roads	.04(9)(b)1.(xi)	3
	(xii)	Proposed final contours	.04(9)(b)1.(xii)	9
	(xiii)	100-year floodplain boundaries	.04(9)(b)1.(xiii)	Fig 3 Operations Manual
	(xiv)	Leachate collection/treatment reservoirs and associated piping	.04(9)(b)1.(xiv)	6
	(xv)	Gas migration control devices	.04(9)(b)1.(xv)	9

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Part E)		Engineering Plans (continued)	TDEC Reference Chapter 0400-11-01, Paragraph	Reference Part E Drawing 10W275-
2.	Detailed diagrams, at a suitable scale, showing:			
	(i)	Sections of erosion and run-on/run-off control structures	.04(9)(b)2.(i)	18, 22, 23, 24
	(ii)	Sections of leachate collection/treatment reservoirs	.04(9)(b)2.(ii)	17,18,19
	(iii)	Sections of gas migration control devices and structures	.04(9)(b)2.(iii)	16
	(iv)	GW monitoring well installations	.04(9)(b)2.(iv)	HydroGeo Report
	(v)	Sections of soil buffer, liner, leachate collection system	.04(9)(b)2.(v)	16
	(vi)	Sections of final cover systems (including required cap)	.03(2)(b)2.(ii) .04(9)(b)2.(vi)	16
	(vii)	Sections of access roads	.04(9)(b)2.(vii)	15
3.	Cross sections (2 per operational area, as a minimum) Scale: 1" = 100' as a minimum showing:			
	(i)	Original ground surface elevations	.04(9)(b)3.(i)	15
	(ii)	Proposed excavation depths	.04(9)(b)3.(ii)	15
	(iii)	Proposed final elevations	.04(9)(b)3.(iii)	15
	(iv)	Soil borings	.04(9)(b)3.(iv)	15
	(v)	Configuration of soil buffer, liner, leachate system; including slopes	.04(9)(b)3.(v)	15
	(vi)	Cells and lifts and associated berms and dikes, and on-site roadways	.04(9)(b)3.(vi)	15
	(vii)	Configuration of final cover system	.04(8)(c)3.(i) .04(9)(b)3.(vii)	15
	(viii)	Configurations of any gas migration control features	.04(9)(b)3.(viii)	9

PART A
OPERATIONS MANUAL

SCEPTER, INC. – HUMPHREYS COUNTY,

PART II PERMIT APPLICATION
SCEPTER, INC. DISPOSAL FACILITY – EAST PHASE SITE
OPERATIONS MANUAL

Prepared for

Scepter, Inc
Waverly,

July 1, 2016
April 5, 2018 Rev 2

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TABLE OF CONTENTS

FIGURES

1.0 INTRODUCTION

This Operations Manual has been developed in support of the Part II Permit application for a new on-site solid waste disposal facility required for Scepter, Inc. to continue disposing of aluminum waste byproducts generated by the company. The proposed disposal facility is identified as the East Phase site, and is located immediately east of the existing landfill.

This Operations Manual has been developed in accordance with rules published by the Tennessee Department of Environment and Conservation (TDEC), specifically Rule 0400-11-01-.04. Scepter is requesting that TDEC issue a permit for the construction and operation of the proposed disposal facility as a Class II facility.

Information presented in this document is organized to be consistent with the permit application requirements of Rule 0400-11-01-.04(9). Sections within this application have been titled and enumerated corresponding to the regulations to aid in the review process.

This Operations Manual is one of several required permit documents completed in support of the Part II permit application. The permit application has been organized into the following Parts as listed below:

- Part A: Operations Manual;
- Part B: Closure/Post-Closure Plan;
- Part C: Groundwater Monitoring Plan;
- Part D: Construction Quality Assurance Plan;
- Part E: Engineering Plans; and
- Part F: Design Calculations.

Additionally, this Operations Manual references the Scepter, Inc. Disposal Facility – East Phase Site Hydrogeological Evaluation report of the Proposed Facility (herein, Hydrogeologic Report). The Hydrogeologic Report was submitted to TDEC under separate cover.

The following italicized text is copied verbatim from the Tennessee Division of Solid Waste Management Rule 0400-11-01-.04. For clarity of discussion, each response is provided in bold text.

2.0 NARRATIVE DESCRIPTION OF THE FACILITY AND OPERATION

Rule 0400-11-01-.04(9)(c). The Part II permit application must include, with appropriate references to the engineering plans and hydrological report, a narrative clearly identifying the following issues:

2.1 FACILITY OPERATOR

Rule 0400-11-01-.04(9)(c)1. Identifies the name of the individual responsible for operation and maintenance of the facility;

The primary contact person responsible for operation and maintenance of the facility is:

Mr. Brian Griffin
General Manager
Scepter, Inc.
1485 Scepter Lane
Waverly, Tennessee 37185
(931) 535-3565

2.2 LOCATION OF THE FACILITY

Rule 0400-11-01-.04(9)(c)2. Describes the location of the facility using roads and highways;

This landfill is located on property owned by Scepter, Inc. near New Johnsonville, Humphreys County, Tennessee.

The site can be reached by taking Exit 143 North from 1-40 West. Proceed north on Highway 13 to Waverly, then west on Highway 70, and then north on Scepter Road to the guard shack at Scepter. The site lies past and to the left of the guard shack.

2.3 BUFFER ZONES

Rule 0400-11-01-.04(9)(c)3. Describes compliance with all applicable buffer zone standards listed in paragraph (3) of this Rule. Each buffer zone standard must be specifically addressed referencing the closest property line, residences, wells, and bodies of water as appropriate, and maps may be attached for easy descriptions and references or otherwise demonstrate compliance.

Rule 0400-11-01-.04(3)(a) Class I and II Disposal Facilities must be located, designed, constructed, operated and maintained such that the fill areas are, at a minimum:

100 feet from all property lines;

As shown on the drawing P-3, the limits of fill are more than 100 feet from the closest property lines.

500 feet from all residences, unless the owner of the residential property agrees in writing to a shorter distance;

The site is located on the Scepter property within which there are no residences. The nearest residences are located greater than a mile to the north on Winsteria Road. Refer to the site Hydrogeologic Report for a regional view of the area.

500 feet from all wells determined to be down gradient and used as a source of drinking water by humans or livestock;

As shown on the Hydrogeological Report, the limits of fill are surrounded on all sides by a buffer greater than 500 feet with respect to down gradient wells used as sources for drinking water. The Hydrogeologic Report provides the location of the nearest drinking water wells.

200 feet from the normal boundaries of springs, streams, lakes, (except that this standard shall not apply to any wet weather conveyance nor to bodies of water constructed and designed to be part of the facility); and

Adjacent springs, streams, and lakes are identified in the Hydrogeological Report along with the limits of waste placement.

There is a stream north of the fill area but it is not within 200 feet of any fill area. The Tennessee River (Kentucky Lake) is not within 200 feet of any fill area. There are several surface water drainage features (wet weather conveyances) and sediment ponds that are within 200 feet of the fill areas, but that is allowed by Rule 0400-11-01-.04-3(a)4.

An unnamed intermittent stream has been delineated within or adjacent to the project area as shown on the drawings which will be protected during landfill construction. One intermittent stream is located more than 200 feet from the limits of fill and will not be disturbed by landfill construction.

Scepter will acquire an Aquatic Resource Alteration Permit from the Tennessee Division of Water Pollution Control (TDWPC) and an Individual Permit (IP) from the Army Corps of Engineers (COE) for impacts from a road crossing to the unnamed intermittent stream within the project site.

A total site buffer with no construction appurtenance within 50 feet of the property line.

For the purposes of buffer discussion, refer to Drawing P-3. Adjacent property lines are depicted along with all the construction elements associated with the facility. As shown on the drawing, construction appurtenances are located more than 50 feet from the closest property.

2.4 FAULT AREAS

Rule 0400-11-01-.04(9)(c)4. Describes its compliance with applicable siting requirements for fault areas.

Rule 0400-11-01-.04(2)(u) New Class I and II SWLF units and lateral extensions shall not be located within 200 feet (60 meters) of a fault that has displacement in Holocene time unless the owner or operator demonstrates in the Narrative Description of the Facility and Operations Manual that an alternative set back distance of less than 200 feet (60 meters) will prevent damage to the structural integrity of the SWLF unit and will be protective of human health and the environment.

Based on a review the United State Geological Survey (USGS) website which contains information on faults and associated folds in the United States that are believed to be sources of M>6 earthquakes during the Quaternary (the past 1,600,000 years including Holocene Epoch), there are no known faults of this age located within the State of Tennessee. The database is intended to be the USGS's archive for historic and ancient earthquake sources used in current and future probabilistic seismic-hazard analyses. This website is a single source that summarizes important information on paleoseismic (ancient earthquake) parameters. These data are compiled from thousands of journal articles, maps, theses, and other documents, as referenced therein. This database is currently located at:

http://earthquake.usgs.gov/hazards/qfaults.

2.5 SEISMIC IMPACT ZONES

Rule 0400-11-01-.04(9)(c)5. Describes its compliance with applicable siting requirements for seismic impact zones.

Rule 0400-11-01-.04(2)(v) Class I and II disposal facilities shall not be located in seismic impact zones, unless the owner or operator demonstrates that all containment structures, including liners, leachate collection systems, and surface water controls systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site. The owner or operator must place the demonstration in the Narrative Description of the Facility and Operations Manual.

Pursuant to the Code of Federal Regulation (CFR) Title 40 PART 258: Criteria For Municipal Solid Waste Landfills, Seismic impact zone means an area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years. The applicable seismic hazard map was originally published in 1991 by USGS (see Figure 1) and has since been updated by USGS and last published in 2008 (see Figure 2). Please note that the return period of 2% in 50 years is essentially equivalent to 10% in 250 years.

As depicted in Figure 2, the site is located in a seismic impact zone. Consequently, the landfill has been designed to withstand the seismic forces occurring during a probabilistic earthquake. The supporting seismic design calculations are included in Part F, Appendix F.4 of the permit application.

2.6 UNSTABLE AREAS

Rule 0400-11-01-.04(9)(c)6. Describes its compliance with applicable siting requirements for unstable areas.

Rule 0400-11-01-.04(2)(w) Owners or operators of Class I and II disposal facilities located in an unstable area must demonstrate that engineering measures have been incorporated into the SWLF units designed to ensure that the integrity of the structural components of the SWLF unit will not be disrupted. The owner or operator must place the demonstration in the Narrative Description of the Facility and Operations Manual operating record. The owner or operator must consider the following factors, at a minimum, when determining whether an area is unstable:

(Unstable area means a location that is susceptible to natural or human-induced events or forces capable of impairing the integrity of some or all of the landfill.)

1. On-site or local soil conditions that may result in significant differential settlings;

The on-site and local soil conditions within and around the landfill footprint were reviewed for susceptibility to the following: liquefaction, excessive or differential settlement, and slope failure. A subsurface exploration program was undertaken at the site as part of the permitting process which characterized the materials beneath the proposed facility to support liquefaction, settlement, and slope stability analyses. This exploration program is further detailed in the site Hydrogeologic Report.

Based on the design of the landfill, the underlying soil conditions do not indicate construction of the landfill will result in significant differential settling. Calculations related to the potential for liquefaction, settlement, and slope stability are provided in Part F, Appendix F.4 of the permit application.

2. On-site or local geologic or geomorphologic features; and

The on-site and local geologic and geomorphic features within and around the landfill footprint were reviewed for the presence of the following: karst terrain (including sinkholes, solution cavities, caves, etc.) and natural areas susceptible to landslides. A subsurface exploration program was undertaken at the site as part of the permitting process which characterized the materials beneath the proposed facility to assess the presence of karst terrain and geological features susceptible to landslides. This exploration program is further detailed in the site Hydrogeologic Report.

Review of the exploration program shows no soil or bedrock characteristics exhibiting low strength, preferential failure planes or fissures, or other characteristics associated with mass movement. Therefore, based on the location of the landfill, no areas which indicate the presence of geologic or geomorphic features susceptible to landslides were detected.

3. On-site or local human-made features or events (both surface and subsurface).

There are no on-site or local human-made features or events of any significance within and around the landfill footprint. No evidence of surface or underground mining exists at the site.

2.7 ACCESS CONTROL

Rule 0400-11-01-.04(9)(c)7. Describes the barriers, signs, procedures and other measures to be used to control access to and use of the facility;

Rule 0400-11-01-.04(2)(b) Control of Access and Use.

1. The facility must have an artificial or natural barrier which completely surrounds the active portion of the facility and must have a means to control entry, at all times, through the gate or other entrances to the active portion of the facility.

The plant site and landfill can only be accessed by proceeding past the guard shack at Scepter's main gate, which is staffed 24 hours a day. Traffic must pass through an additional gate for access to the landfill site. Although the entire site boundary is not currently fenced, adequate fences and other prohibiting barriers have been installed as deemed necessary to control unauthorized access to, or use of, the site.

To further control access when the site is not in use, the access gate to the landfill will be locked. The guards will control access to the landfill, thus preventing unauthorized access and use. Keys to the gate will be provided to authorized personnel only.

The material intended for disposal will consist of nonhazardous waste material from secondary aluminum rotary furnace operations (salt cake, baghouse dust, cooling tower sludge, and refractory furnace brick). Authorized waste will include waste from Scepter's Operations.

1. If open to the public, the facility must have clearly visible and legible signs at the points of public access which indicate the hours of operation, the types of materials that will or will not be accepted, emergency telephone numbers, schedule of charges, and any other necessary information.

The facility is a captive facility owned and operated by Scepter and is not open to the public. Therefore, this regulation is not applicable. However, a sign is positioned at the entrance to the facility identifying the site and stating that unauthorized access and use are not permitted. Information to be provided on the sign is:

Name of Entity Served:	SCEPTER, INC.
Emergency Phone:	931/535-3565
Acceptable Waste:	PROCESS WASTE AND REFRACTORY FURNACE BRICK

2. If the facility is open to the public, or if it is otherwise necessary for proper operation, roads within the facility, easement, and parking areas shall be designed, constructed, and maintained so as to be accessible in all weather conditions. Traffic control signs shall be provided as necessary to promote an orderly traffic pattern to and from the solid waste discharge area to maintain efficient operating conditions.

The facility is a captive facility owned and operated by Scepter and is not open to the public. Transportation of wastes from the Plant will be made along existing haul roads running along the perimeter of the facility. The access road to the leachate collection tanks will be designed, constructed, and maintained so as to be accessible in all weather conditions. Traffic control signs will not be necessary because the site will have controlled access and will be used by Scepter personnel only.

Periodic maintenance of these roads will be completed as part of the operation of the facility. Maintenance will include re-grading the surface, adding gravel as needed, and cleaning roadside ditches to improve or provide drainage. Operational access roads or ramps will be constructed as needed from the perimeter road to the disposal fill area to provide access to the working face. A temporary turnaround area will be constructed at the working face of the facility to provide for proper unloading of the trucks.

3. The facility must have trained personnel present and on duty at all times it is in operation to assure compliance with operational requirements and to prevent entry of unauthorized wastes.

The facility is a captive facility owned and operated by Scepter and is not open to the public. Trained personnel will be present and on duty whenever solid waste is being transported, deposited, or covered. Their duty will be to assure compliance with operational requirements and to prevent disposal of unauthorized wastes.

4. There must be no scavenging at the facility. Any salvaging or recycling operations must be conducted at safe, designated areas (not working face) and times, and in a sanitary manner.

Scavenging is not permitted at the facility, and is not anticipated due to the nature of the waste disposed at the facility. Any recycling of the material will be conducted at safe, designated areas and times, and in a sanitary manner.

2.8 METHODS AND SEQUENCE OF OPERATIONS

Rule 0400-11-01-.04(9)(c)8. Describes the methods and sequence of operations;

The disposal facility will be developed in a series of cells through four phases over the life of the landfill. Phases - 1, 2, and 3 will be constructed sequentially moving west to east as additional disposal capacity is needed. Phase 4 will then be constructed to the north and over existing Phases 2 and 3. Each cell will be constructed with a composite liner system as depicted on Drawing P- 17. Leachate will be collected at the low point within the waste limits. Once waste is placed within a cell, any water collected by the leachate collection and removal system will be considered leachate and will be pumped to storage tanks prior to discharge under an existing NPDES permit. Temporary storm water diversion structures, including ditches, rain flap berms, sumps, and pumps will be used as needed to minimize storm water run-on into the active portion of a cell. Non-contact storm water will be diverted to the sediment basin. The active area of any cell will be minimized to reduce leachate generation and the potential for fugitive dust emissions. In general, active areas will be limited to a maximum working area of approximately 5 acres.

The general sequence of construction is depicted in Drawings P-10 through P-13. Initial construction will consist of a cell within Phase I, the construction of the Sediment Basin, and construction of a haul road to provide access from the plant to the disposal facility. As cells are developed, future cell areas will be utilized as temporary stockpile, laydown and borrow areas.

Final top of waste placement grades are depicted on Drawing P-8. Surface water drainage features and either intermediate cover or the final cover system will be installed progressively as filling proceeds.

2.9 Types and Anticipated Volumes of Solid Wastes

Rule 0400-11-01-.04(9)(c)9. Describes the types and anticipated volumes of solid wastes to be disposed of and the sources which generate the waste, and for special wastes, the physical and chemical characteristics of the wastes and any special handling procedures to be utilized;

This landfill is intended to receive only waste from secondary aluminum processing operations consisting of waste cake material, baghouse dust, cooling tower sludge, and waste furnace brick from aluminum furnaces only.

It is not the intent of Scepter, Inc. to accept waste from non-secondary aluminum processing facilities, nor does Scepter intend to dispose of waste material other than those identified as a result of the secondary aluminum production process. Only upon written approval of the Tennessee Division of Solid Waste Management (DSWM) would other special wastes be disposed in this landfill.

The salt cake and baghouse dust are generated during the recovery of aluminum from scrap receipts. The waste materials generated by Scepter will be stored inside a building until they are transported, placed, compacted and covered in the proposed landfill or, weather permitting, these materials may be transported to the landfill for unloading, placing, compacting, and covering. In either case, all salt cake will be allowed to cool prior to landfill disposal.

Periodically the refractory furnace brick lining material in these secondary aluminum processing facilities must be replaced. The amount of refractory lining fill material, in comparison to the salt cake being generated by a given furnace, is relatively minor.

The primary wastes generated by Scepter are salt cake at approximately 90% of the volume and baghouse dust being the other 10%. These are wastes consisting of sodium and potassium salts, aluminum oxides, ammonia, and other trace metal oxides.

Table 1. Summary of Waste Disposal Volumes

Waste Material	Tons/Year	Tons/Day	Cubic Yards/Year	Cubic Yards/Day
Salt Cake	59,000	160	52,500	144
Baghouse Dust	500	1.5	1,500	4
Other[1]	500	1.5	500	1.5
Total	60,000	163	54,500	150

[1] Other process wastes include cooling tower sludge or refractory brick

As shown in Table 1, the facility is anticipated to receive approximately 54,500 cubic yards of secondary aluminum waste annually. The estimated quantity of material to be generated is based on empirical data and may vary from these estimates due to prevailing economic conditions for the aluminum recycling industry. Future changes to the technology utilized in the plant process may also impact the estimated annual quantities of generation.

The East Phase Site will provide storage volume for an estimated 2.85 million cubic yards of waste (including daily cover soils). The top of waste placement grades are provided on Drawing P-8 of the Engineering Plans. Based on the anticipated generation rates presented in Table 1, the facility provides disposal capacity for approximately 43 years.

2.10 Number of Acres to be Filled and Phase Development

Rule 0400-11-01-.04(9)(c)10.Identifies the number of acres to be filled and the total number of acres to be permitted, including buffer zone acreage (Note: If the site is to be developed in accordance with a phased development plan, each parcel must be separately addressed. If minimum closure areas are to be utilized such proposal must be described here and delineated in the closure plan);

The limits of waste placement are depicted on the Engineering Plans provided in Part E of the permit application. The total fill area encompassed by the limits of waste placement is 26.4 acres. The landfill will be developed in four phases. The phased development of the landfill cells are depicted on Drawings P-10 through P-13. Table 2 provides a summary of the area and estimated volume of waste material that will be placed during the development of each Phase.

Table 2. Summary of Waste Disposal Capacity

Phase Development	Waste Placement Area (Acres)	Disposal Capacity (Cubic Yards)	Anticipated Life (Years)
Phase 1	7.49	284,836	4.4
Phase 2	8.66	836,104	12.8
Phase 3	7.23	1,096,353	16.8
Phase 4	3.02	635,304	9.7
Total	26.4	2,852,597	43.7

The top of waste placement grades are provided on Drawing P-8 of the Engineering Plans. Final cover grades of the landfill are shown on Drawing P-9.

2.11 Waste Handling and Covering Program

Rule 0400-11-01-.04(9)(c)11. Describes the waste handling and covering program, to include but not necessarily be limited to, descriptions of:

(i) Unloading, spreading, and compacting operations;

Wastes will be transported and conveyed to the landfill working face using heavy duty dump trucks for salt cake and hydraulic dump rolloffs for baghouse dust contained in supersack bags. Trucks will use access roads contained within the facility. Once placed, the waste will be spread and compacted utilizing the landfill equipment maintaining a minimum thickness of three feet of material between the equipment and the underlying geosynthetics liner.

The placement of solid waste will be confined to the smallest workable area and will be spread and compacted with appropriate equipment owned or leased by Scepter.

- Once a final grade is achieved, a final cover shall be placed as depicted in the engineering plans.
- Final waste slopes will not exceed 3H:1V.

(ii) The frequencies and depths of initial, intermediate, and final cover; and

- Waste will be covered with a daily cover consisting of six (6) inches of native soil to be uniformly compacted. The waste will be covered to minimize potential releases of solid wastes or their constituents, and to control disease vector. The waste material is not believed to be conducive to the harborage or propagation of disease vectors. An intermediate cover will be placed on the side slopes and vegetated to reduce infiltration of surface waters as the facility's height progresses and final grades are reached. This periodic placement of an intermediate cover will prevent erosion and help maintain the integrity of the disposal facility side slopes. The intermediate cover will consist of a 12-inch-thick layer of compacted soil (that includes the 6 inch daily cover). This compacted soil shall be seeded with appropriate vegetation.

Final cover will be placed in accordance with details shown on Drawing P-17 when the facility reaches final waste placement grades. Final closure will be completed as described in the Closure/Post-Closure Plan provided in Part B.

(iii) The cover material(s) to be utilized, including the estimated volumes to be needed (show initial, intermediate, and final earthwork calculations) and their sources and availability (Also see Rule 0400-11-01-.04(2)(h)).

The estimated quantities of interim/Intermediate and final cover soil are summarized in Table 3.

TABLE 3. SUMMARY OF COVER MATERIAL QUANTITIES

COVER MATERIAL	THICKNESS (INCHES)	QUANTITY (CUBIC YARDS)
Daily Cover	6	475,000
Intermediate Cover	12	44,500
Barrier Cover Soil	6	22,300
Final Cover- Protective Cover Soil	18	66,800
Final Cover - Vegetative Cover Soil	6	22,300

Excess soil material excavated during landfill construction will be stockpiled in designated borrow/stockpile areas. Soil balance estimates indicated that sufficient materials will be available from on-site sources. Where on-site soil is insufficient in terms of quantity or quality, Scepter may supplement on-site soil with off-site borrow materials provided the soil meets the requirements contained in the Construction Quality Assurance (CQA) Plan provided in Part D.

2.12 LANDFILL OPERATIONS EQUIPMENT

Rule 0400-11-01-.04(9)(c)12. Describes the operation equipment to be utilized (including back-up equipment), and their source and availability;

Rule 0400-11-01-.04(2)(g) Operating Equipment – At Class I disposal facilities, and at Class II, Class III and Class IV disposal facilities unless the Commissioner deems some other arrangement as adequate for proper facility operation, there must be maintained on-site operating equipment capable of spreading and properly compacting the volume of solid wastes received, and capable of handling the earthwork required. Back-up equipment must be available within 24 hours of primary equipment breakdown.

Operating Equipment

Operating equipment will be maintained and operated by Scepter personnel or contractors hired and trained by Scepter. The waste which is not directly transported to the landfill, will be stored under cover to prevent exposure to weather related events and then transported to the landfill periodically.

Equipment necessary for operations includes typical earthmoving equipment as listed below.

- Excavators
- Hauling Trucks
- Bulldozers
- Loaders
- Water Trucks

Scepter uses, or will allocate to this facility, equipment that is adequate in both size and quantity for transport and placement of the waste materials. Scepter can also rent or lease back-up equipment within 24 hours of primary equipment breakdown.

2.13 LITTER CONTROL

Rule 0400-11-01-.04(9)(c)13. Describes the structures and procedures to be used in controlling and collecting litter;

Rule 0400-11-01-.04(2)(d) A facility must be operated and maintained in a manner to minimize litter. Fencing, diking and/or other practices shall be provided as necessary to confine solid wastes subject to dispersal. All litter must be collected for disposal in a timely manner;

The solid waste contains no litter that is subject to dispersal by the wind. Therefore, blowing litter should not be a problem. Measures such as fencing, dikes, etc., will be implemented to confine the solid waste, if necessary.

2.14 RUN-ON, RUN-OFF, AND EROSION CONTROL

Rule 0400-11-01-.04(9)(c)14. Describes how run-on and run-off collection and holding and erosion control facilities will be managed, including the disposition of collected waters and residues and a comparison of before and after flows in the drainage ways leaving the site;

Rule 0400-11-01-.04(2)(i) Run-on, Run-off, and Erosion Control

1. The operator must design, construct, operate, and maintain a run-on control system capable of preventing flow onto the active portion of the facility for all flow up to and including peak discharge from a 24-hour, 25-year storm.

The engineering plans show and describe diversion ditches and other drainage ways intended to prevent flow onto the active portion of the landfill. This includes all flow up to and including the peak discharge anticipated from a 24-hour, 25-year storm event. This flow will be non-contaminated and directed away from the leachate collection system. The flow resulting from drainage from the area outside of the leachate collection area will be directed around the active portion and into the sediment pond.

Run-on from adjacent land is diverted away from the landfill by existing natural drainage features. The landfill facility rests within the confinement of natural ridgelines that do not permit run-on to the site.

Design calculations for the surface water management features are presented in Part F, Appendix F.1 of the permit application.

2. The operator must design, construct, operate, and maintain a run-off management system to collect and control at least the peak flow resulting from a 24-hour, 25 year storm.

The diversion ditches will be relocated as necessary as the active portion progresses. The remaining ditches shall be maintained as necessary. The sediment pond will be checked periodically for erosion and sediment accumulation. Sediment will be removed as necessary to maintain the required storm water volume.

Permanent run-off management features are shown on Drawing P-14. Associated details are provided on Drawings P-15, P-21, P-22 and P-23. The outer slopes of the facility have been designed with terraces spaced every 95 feet of slope length (30 feet vertical spacing) that will be constructed progressively as the elevation of the disposal facility is raised. Rock-lined letdowns will convey surface water run-off by gravity to perimeter channels at the base of the landfill. Surface water runoff will be collected in a sediment basin prior to discharge. All permanent run-off measures, including perimeter channels and culverts, are designed to collect and control the peak flow resulting from a 25-year/24-hour storm under final design conditions.

Design calculations for the surface water control structures are contained in Part F, Appendix F.1 of the permit application.

3. Holding facilities (e.g. sediment basins) associated with run-on and run-off control systems must be designed to detain at least the water volume resulting from a 24-hour, 25 year storm and to divert through emergency spillways at least the peak flow resulting from a 24-hour, 100 year storm.

The sediment pond has been designed to detain the volume resulting from a 24-hour, 25-year storm (dry storage volume) plus 67 cubic yards per acre of drainage area for sediment storage (wet storage volume). The pond has an emergency spillway designed to pass the peak flow resulting from a 24-hour, 100-year storm.

Holding facilities associated with storm water management consist of a sedimentation basin as depicted on Drawing P-15. Details for the sediment basins are provided on Drawing P-23.

Design calculations for the surface water control structures are contained in Part F, Appendix F.1 of the permit.

4. Collection and holding facilities associated with run-on and run-off control systems must be emptied or otherwise managed expeditiously after storms to maintain design capacity of the system.

Surface water collection facilities will be open drainage ways and thus will empty themselves after storms to maintain design capacity. The sediment pond has a vertical riser pipe with holes drilled into it. These holes are designed to allow the sediment pond to drain slowly and thus present a drained pond within 72 hours for the next storm event.

The sediment basin has been designed with a low-flow device to empty the sediment basin following rain events, in accordance with design standards provided in the Tennessee Erosion and Sediment Control Handbook [TDEC, 2002]. However, in the event that the low flow device becomes clogged or does not have adequate time to dewater the basins prior to a storm event, the primary spillway of the sediment basin has been designed to manage runoff resulting from the 25-year/24-hour storm without the function of the low-flow device. Similarly, the sediment basins' primary and emergency spillways are designed to manage runoff resulting from the 100-year/24-hour storm event without considering the function of the proposed dewatering devices. Sediment basins will be cleaned periodically to maintain the minimum required storage volume.

Design calculations for the surface water control structures are contained in Part F, Appendix F.1 of the permit.

5. Run-on and run-off must be managed separately from leachate unless otherwise approved by the Commissioner.

The run-on and run-off diversion systems are separate from the leachate collection, transportation, and storage facilities.

As shown in the engineering drawings, the leachate and storm water are designed to be handled separately. Leachate will be collected in perforated collection pipes and pumped by slope riser pumps via a force main to storage tanks. Storm water will be collected and managed by a sediment basin as described in this section.

As filling progresses, intermediate cover soil will be placed on all outboard slopes prior to allowing runoff to enter the storm water management system. Water that comes in contact with waste will be treated as leachate and collected in the facility's leachate collection system.

6. The operator must take other erosion control measure (e.g., temporary mulching or seeding, silt barriers) as necessary to control erosion of the site.

Hay bale fences and check dams will be used to control erosion. In addition, the slopes of cuts and fills have been designed with the objective of reducing erosion. Completed areas will be seeded and mulched as often as needed to provide a vegetative cover to help prevent erosion.

Storm run-off will be controlled and managed through a series of temporary and permanent surface water and erosion control measures including silt fences, seeding and mulching, and drainage channels. A Storm Water Pollution Prevention Plan will be developed to obtain coverage under the NPDES Construction General Permit for storm water discharge from construction activities.

The following guidelines will be followed in establishing final cover unless required otherwise by the plans or the Engineer:

Seeding

Seeds shall meet the requirements of applicable specifications provided below. Each variety of seed shall be furnished in separate bags. Each bag shall be labeled to show the variety, weight, purity, germination, and test data prescribed by law. All test results shall be fully certified by the vendor or by a recognized seed testing agency. Seed shall contain a weed seed content no higher than 0.25 percent and shall be free of noxious weeds.

Scepter reserves the right to require that samples be furnished, and to inspect and test the seeds after delivery. Seeds found not to comply with specification requirements shall be subject to rejection. Seeds shall not be mixed until each variety of seed to be used in the mix has been inspected and/or tested separately and approved.

Permanent seeding shall be performed between March 15 and May 15 or between August 15 and October 15. At other times, sodding or seeding with temporary seed shall be made until the appropriate spring or late summer permanent seeding time. Table 4 provides a summary of recommended seed blends and application rates to be used for each seeding season.

TABLE 4. RECOMMENDED SEED BLENDS AND APPLICATION RATES

SEASON	SEED	APPLICATION RATE (POUNDS PER ACRE)
Spring (March 15 – May 15)	Kentucky 31 Fescue	50
	White Clover	25
	Weeping Lovegrass	15
Summer (May 15 – August 15)	Bermuda Grass (hulled)	50
Fall (August 15 – October 15)	Kentucky 31 Fescue	60
	White Clover	15
Winter (October 15 – March 15)	Annual Ryegrass	50
	White Clover	10

Prior to seeding, soil shall be prepared by establishing approved grades, and removing deleterious material, such as large roots, stones, or debris which may impede mowing. Seed shall be hydraulically applied at the application rate specified in Table 4 using hydroseeding equipment. Seeding shall not be performed on frozen or muddy grounds, or when prevailing winds exceed five miles per hour.

Fertilization

Fertilizer used shall be a commercial or agricultural grade with a 17% nitrogen, phosphorous, and potassium content (17-17-17), or approved equal 1:1:1 ratio fertilizer. Fertilizer shall be free-flowing, uniform in composition, and shall conform to

State and Federal regulations. Fertilizer shall bear the manufacturer's statement of analysis. The rate of application for fertilizer in permanent seeding applications shall be 1,000 pounds per acre, or as specified in the plans. Fertilizer may be applied with equipment capable of providing even spreading of the fertilizer over the entire seeded area.

Mulching

Mulching shall be either fiber mulch or straw mulch. Application of mulch will take place no more than 24 hours after seeding of the area.

Fiber mulch shall consist of a biodegradable, dyed-wood, cellulose-fiber mulch that is nontoxic and free of plant-growth or germination inhibitors and has a maximum moisture content of 15 percent and a pH range of 4.5 to 6.5. Fiber mulch shall be applied in a slurry mix using hydroseeding equipment. The material shall be applied over the seeded area in a manner not disruptive to the placement of seed.

Straw mulch shall consist of an air-dried, clean, mildew- and seed-free, salt hay or threshed straw of wheat, rye, oats, or barley. Straw mulch shall be free of noxious weeds, weed seeds, sticks, rocks, or other deleterious material. Straw mulch shall be applied at a rate of 92 pounds per acre with a blower, or other approved equipment capable of distributing the material evenly over the seeded area without disrupting the placed seed.

2.15 Leachate Migration Control Standards

Rule 0400-11-01-.04(9)(c)15. Describes how leachate collection and holding facilities will be managed, including the disposition of collected leachate;

Also refer to Rule 0400-11-01-.04(4) Leachate Migration Control Standards

The landfill facility will be constructed with a composite liner system as shown on Drawing P-16. An alternative liner system has been proposed for the facility consisting of the following components, in the order of construction:

- A two foot thick compacted clayey soil layer with a maximum permeability of 1×10^{-6} cm/sec;
- A geosynthetic clay liner (GCL);
- A 60-mil thick, textured high density polyethylene (HDPE) flexible membrane liner (FML);
- A double-sided geocomposite drainage layer supplemented with 6-inch diameter perforated HDPE leachate collection pipes; and
- Two feet of protective cover to protect the liner system and provide a filter between the geocomposite drainage layer and the waste material. The protective cover will consist of 1 foot of sand, and 1 foot of native soils. A graded filter

calculation is provided in Appendix F.2. The calculation provides minimum grain size distribution requirements for the sand component of the protective cover material which will be in contact with the geosynthetics below and the native soil protective cover above.

These materials will have the appropriate chemical properties and sufficient strength and thickness to prevent failure due to pressure gradients, physical contact with the waste or leachate to which they may be exposed, climatic conditions, the stress of installation, and the stress of daily operations. The liner system is to be placed upon a foundation of native soil which is capable of providing support to the liner system and resisting any pressure gradients from above and below the liner to prevent failure of the liner system due to settlement, compression, or uplift.

The leachate collection system design is depicted on Drawing P-6. Details of the leachate collection system are provided on Drawings P-18, P-19 and P-20. The leachate collection system is designed to facilitate the free drainage of leachate from the base of the landfill to a low point, or sump. A system of perforated leachate collection pipes is designed at a minimum 1% slope to maintain a positive grade to the leachate collection sump of each cell. Cleanout risers will be provided for each leachate collection pipe lateral to facilitate the cleaning and maintenance of the leachate collection system. The geocomposite drainage layer and leachate pipe spacing is designed to collect expected leachate generation rates while maintaining flow within the geocomposite drainage layer and therefore maintaining less than 1 foot of head on the liner system.

Leachate will be removed from the leachate collection sumps by a system of leachate collection extraction pumps contained within side slope riser pipes. The leachate extraction pumps will operate in a lead-lag configuration. Leachate will be extracted from the cell's leachate collection system and pumped through a leachate transfer force main. Valves and access manholes are provided to allow maintenance and repairs to pump and piping systems.

Leachate collected at the facility will be transferred by a dual containment force main to storage tanks and disposed via an NPDES permit in accordance with all applicable state and federal regulations. Leachate generation is expected to be produced on the order of 60,000 gallons based on a 30 day average rate as predicted by the USEPA HELP Model. Regarding the regulatory 30 day storage capacity requirement, Scepter is providing sufficient storage capacity to meet the requirement as follows: A) A new tank system is proposed to provide a storage capacity of 30,000 gallons B) The existing 'west phase' landfill leachate capacity is provided by two tank systems totaling 40,000 gallons of storage. The west phase is nearing the end of its operational life, so these tanks will be available for storage in the future. These tanks will be connected to the East Phase collection system if necessary. C) Scepter has a site specific NPDES permit that allows up to 5,000 gallons per day of leachate disposal in addition to its aluminum casting production wastewater. Such leachate is currently transported in a 2,500 gallon tanker truck.

In the event where an NPDES permit becomes ineffective, Scepter will contact a third party for management of the material off-site.

Calculations supporting the design of the leachate collection system are provided in Part F, Appendix F.2 of the permit application.

2.16 DUST CONTROL

Rule 0400-11-01-.04(9)(c)16. Describes the dust control measures to be taken and when they would be implemented;

Rule 0400-11-01-.04(2)(j) The operator must take dust control measures as necessary to prevent dust from creating a nuisance or safety hazard to adjacent landowners or to persons engaged in supervising, operating, and using the site. The use of any dust suppressants (other than water) must be approved in writing beforehand by the Department.

It is not anticipated that dust will become a problem as the site is remote and there are no close neighbors. However, dust control measures to prevent creation of a nuisance or safety hazard to adjacent landowners or to persons engaged in supervising, operating, and using the site will be initiated as necessary by applying water via a water truck to active access roads.

2.17 FIRE SAFETY

Rule 0400-11-01-.04(9)(c)17. Describes the fire safety precautions and procedures to be taken, the types and availability of on-site fire suppression equipment, and/or the arrangements made with the local fire protection agency;

Rule 0400-11-01-.04(2)(c) Fire Safety

1. Except as may be specifically authorized by the Department:

   (i) The operator must not permit or engage in open burning of solid wastes at the facility. Any open burning that does occur must be immediately extinguished.
   (ii) The operator must not allow solid wastes which are burning or smoldering to be deposited into the active portion of the facility. Any such wastes that are received must be deposited at a location safely removed from the active portion and extinguished before being deposited into the active portion.

No open burning of solid wastes will be allowed at this facility. The nature and characteristics of the solid wastes to be disposed at this site will not support combustion. Since the solid waste is cake from the furnace(s), it is possible that the solid waste may be at an elevated temperature when removed from the furnace. The solid waste will be allowed to cool to safe temperatures before being placed in the landfill facility.

2. The facility must have, on-site and continuously available, properly maintained fire suppression equipment in sufficient quantities to control accidental surface fires that may occur, or arrangements must be made with the local fire protection agency to provide immediate fire fighting services when needed. Additional earth moving equipment shall be brought to the facility as necessary to help suppress an underground fire.

The solid waste material is composed of inert non-flammable materials; therefore a fire involving waste materials is very unlikely. In addition, the Humphreys County Fire Department is notified of the waste characteristics/hazards through the planning program of the Emergency Planning & Community Right to Know Act.

2.18 Personnel Services / Communications

Rule 0400-11-01-.04(9)(c)18. Describes the facilities and services available to facility personnel, including shelter, drinking water, handwashing and toilet facilities, and communications equipment;

Also see Rules 0400-11-01-.04(2)(e) Personnel Services and 0400-11-01-.04(2)(f) Communications.

A communications system is not necessary because it is anticipated that the solid waste will be stockpiled under roof and then transported to the landfill periodically. The landfill will be staffed only during transport, disposal, and covering activities. If it becomes apparent that a communications system is required for the safe and effective operation of the landfill, one will be installed.

Shelter, potable water, handwashing, telephones, and restrooms are all available at the main plant buildings just north of the landfill facility.

2.19 Construction Quality Assurance Plan

Rule 0400-11-01-.04(9)(c)19. Describes in a construction quality assurance plan:

(iii) How each new “as built” solid waste landfill unit(s) liner(s) and/or lateral expansion liner(s) and cover system(s) will be inspected and/or tested by a registered engineer as required at rule 0400-11-01-.04(1)(c) during construction or installation for uniformity, damage, and imperfections, and

(iv) How each constructed Section of the liner system or final cover system will be certified by a registered engineer.

A Construction Quality Assurance (CQA) Plan has been developed for supporting construction activities for the site. The CQA plan is contained in Part D of the permit.

2.20 GAS MIGRATION CONTROL STANDARDS

Rule 0400-11-01-.04(9)(c)20. Describes how the migration of explosive gases will be controlled and monitored;

The Engineering plans show a gas migration control system that will be designed, constructed, operated, and maintained so any gases generated by decomposition or other reactions of the solid waste will be collected and vented to the atmosphere. The gas collection system will not allow a build-up of gas pressure that would compromise the final cover.

The concentration of explosive gases at the property boundaries should not exceed the lower explosive limit for gases due to the inert nature of the waste being placed in the landfill.

2.21 GROUNDWATER MONITORING PROGRAM

Rule 0400-11-01-.04(9)(c)21. Describes the planned ground water monitoring program, to include but not necessarily be limited to, descriptions of:

(i) The number and location of wells or other monitoring points;

(ii) Monitoring well construction;

(iii) The parameters to be monitored for and the frequency they will be checked;

(iv) Sampling and analytical procedures and methods to be used; and

(v) How the sampling and analytical results will be recorded and reported to the State.

A Groundwater Monitoring Program as required by Rule 0400-11-01-.04(7) has been developed for supporting groundwater monitoring for the site. It outlines the following:

- The number and location of wells or other monitoring points;
- Monitoring well construction;
- The parameters to be monitored for and the frequency they will be checked;
- Sampling and analytical procedures and methods to be used; and
- How the sampling and analytical results will be recorded and reported to the State.

The Groundwater Monitoring Program Plan is contained in Part C of the permit.

2.22 LOCATION IN FLOODPLAINS

Rule 0400-11-01-.04(9)(c)22. Includes an engineering statement of the site flood frequency exposure and describes flood protection measures to be taken;

Figure 3 shows the limits of the 100 year flood based on the Federal Emergency Management Agency Flood Insurance Rate Maps in comparison to the project site. The landfill site is located outside the flood plain; therefore, no additional flood protection measures are necessary and are not part of the design of the facility.

Rule 0400-11-01-.04(2)(n) Facilities must not be located in a 100-year flood-plain unless it is demonstrated to the satisfaction of the Commissioner that:

1) Location in the floodplain will not restrict the flow of the 100-year flood nor reduce the temporary water storage capacity of the floodplain.
2) The facility is designed, constructed, operated, and maintained to prevent washout of any solid waste.

The site varies in elevation from 400 feet to 620 feet, mean sea level. The Engineering Plans set a maximum cut elevation of 460 feet. The elevation for the 100-year floodplain obtained from the Federal Emergency Management Agency Flood Insurance Rate Maps is 375 feet. The deepest excavation cut will be approximately 85 feet above the 100-year floodplain elevation, therefore based on the landfill's location, no additional flood protection measures are necessary and are not part of the design of the facility.

2.23 ENDANGERED AND THREATENED SPECIES

Rule 0400-11-01-.04(9)(c)23. Describes the impacts the facility will have on endangered or threatened species or plants, fish, or wildlife or their habitat;

AECOM has reviewed the FWS (IPaC) and Tennessee Natural Heritage websites. Federally threatened or endangered (T&E) species listed in Humphreys County, Tennessee in proximity of the project site were reviewed. A total of seven T&E species were listed. These included four mussel species and three bat species. All mussel species are endemic to the Tennessee River and will not be impacted by the project activities. No caves are present on the project site. Areas of marketable timber have been harvested and removed from portions of the project area. All other marketable timber, as well as dead and dying timber will be harvested and removed from the project area during winter months by silviculture practices. Trees with the potential to provide summer roost or hibernacula for bat species will not be present as stages of the landfill development progresses.

Rule 0400-11-01-.04(2)(m) Facilities shall be located, designed, constructed, operated, maintained, closed, and cared for during the post-closure care period in a manner that does not:

Cause or contribute to the taking of any endangered or threatened species of plants, fish, or wildlife; or

Result in the destruction or adverse modification of the critical habitat of endangered or threated species.

See discussion above.

2.24 RANDOM INSPECTION PROGRAM

Rule 0400-11-01-.04(9)(c)24. Describes the random inspection program required under Rule 0400-11-01-.04(2)(s).

Since this facility is a dedicated landfill and will receive only secondary aluminum processing waste from the rotary furnaces, no random inspection program is necessary. The facility personnel will be trained to recognize the "normal" solid waste and will be directed not to receive wastes other than secondary aluminum processing waste. If an unusual or suspect waste is observed at the facility, the personnel will be directed to contact the Scepter General Manager. He, in turn, will contact the field office of the Tennessee Division of Solid Waste Management.

2.25 WASTE RESTRICTIONS

Rule 0400-11-01-.04(2)(k)1. A facility may receive for disposal only those solid wastes it is allowed to manage under the terms of its permit.

The facility is a captive facility owned and operated by Scepter solely to dispose of secondary aluminum processing waste materials. No other wastes will be disposed of at the facility.

2.26 REQUESTED VARIANCES FROM REGULATIONS

Rule 0400-11-01-.01(5). Variances and Waivers - After public notice and an opportunity for public comment, any standard, or requirement in these rules may be waived by the Commissioner if the operator can demonstrate, to the satisfaction of the Commissioner, that the standard is inapplicable, inappropriate, or unnecessary to his facility, or that it is equaled in effect by alternative standards or requirements. Any requests for such requests must include the following:

(a) An identification of the specific requirement(s) for which a waiver is requested;

(b) An explanation of the reason(s) why the requirement(s) should be considered inapplicable, inappropriate, or unnecessary, and/or a description of the alternative procedures or mechanisms to be utilized and why they should be considered equal in effect to the standard(s) proposed to be waived; and

(c) Any other such information as the Commissioner deems necessary for his evaluation of the request.

(d) Any Class I variances or waivers granted will not be less stringent than the standards of 40 CFR 257 and 258 (Solid Waste Disposal Facility Criteria Final Rule, October 9, 1991).

Scepter requests TDEC issue a waiver for the following requirements.

<u>Alternate Liner System</u>

Rule 0400-11-01-.04(4)(a)(1). Such facilities must have a liner designed to function for the estimated life of the site and the post-closure care period. It shall be designed, constructed, and installed to ensure that the concentration values listed in Appendix III of this rule will not be exceeded in the uppermost aquifer at the relevant point of compliance. The liner must be:

(i) A composite liner consisting of two components; the upper component must consist of a minimum 30-mil flexible membrane liner (FML), and the lower component must consist of at least a two-foot layer of compacted soil with a hydraulic conductivity of no more than 1.0 x 10-7 cm/sec. FML components consisting of high density polyethylene (HDPE) shall be at least 60-mil thick. The FML component must be installed in direct and uniform contact with the compacted soil component;

Scepter has proposed an alternate liner system as described in Section 2.15 of this Operations Manual and illustrated in Drawing P-17 for use in this facility. This is the same liner system approved for construction for the Phase 4 expansion of the Western Phase of the landfill. Engineering calculations demonstrating the equivalent or superior performance of the proposed alternate liner system are provided in Part F, Appendix F.2, as required by Rule 0400-11-01-.04(a)(4). Scepter requests TDEC issue a waiver for use of this alternate liner system.

<u>Alternate Final Cover System</u>

Rule 0400-11-01-.04(8)(c)3. Unless otherwise noted in the permit a depth of compacted final cover material (e.g., soil) shall be placed on the disposal facility or disposal facility parcel in the shortest practicable time, not to exceed 90 days, after achieving final grade of any fill area or any portion of a fill area. At least the top twelve inches of this cover material shall be soil which will support the growth of suitable vegetation (e.g., topsoil).

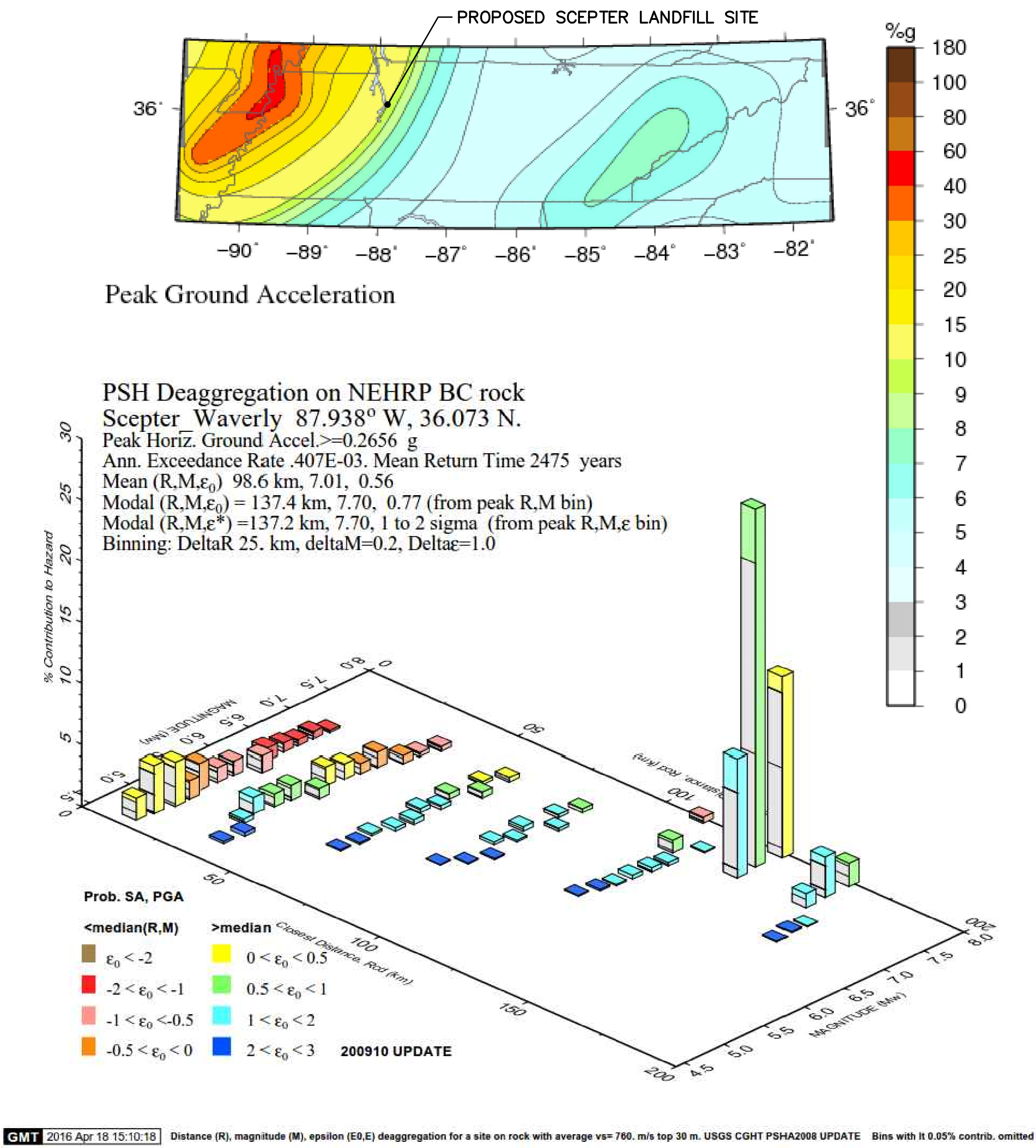
(i) At Class I and Class II facilities the depth of final cover system shall be at least 36 inches of soil of which a minimum of 12 inches shall be for the support of vegetative cover.

The design of the final cover system shall be such that the infiltration volume of water will be equal to or less than the percolation volume through the bottom liner system or a design which includes a compacted soil layer of at least 24 inches which has a permeability no greater than 1 × 10-7 cm/sec, whichever is less. This design shall be supported by the use of the HELP model or other equivalent method approved by the Commissioner.

An alternate final cover system may be used provided that it is demonstrated to the satisfaction of the Commissioner that the final cover system provides equivalent or superior performance to the minimum performance standard in this subpart.

Scepter has proposed an alternate final cover system as described in Section 2.2 of the Closure and Post-Closure Plan in Part B and illustrated in Drawing P-17. This is the same final cover system approved for construction for the Western Phase of the landfill. The proposed alternate final cover system will result in less moisture leakage into the landfill than the TDEC prescribed system. Engineering calculations demonstrating equivalent or superior performance of the proposed alternate final cover system are provided in Part F, Appendix F.3 as required by Rules 0400-11-01-.04(8)(c)3(i) and 0400-11-01-.04(8)(c)3(iii). Scepter requests TDEC issue a waiver for use of this alternate final cover system.

FIGURES



AECOM

SCEPTER, INC.

WAVERLY, TENNESSEE

SEISMIC IMPACT ZONE 2% PROBABILITY OF EXCEEDANCE IN 50 YEARS

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	FIGURE
NP	MW	60398526	7/1/16	1

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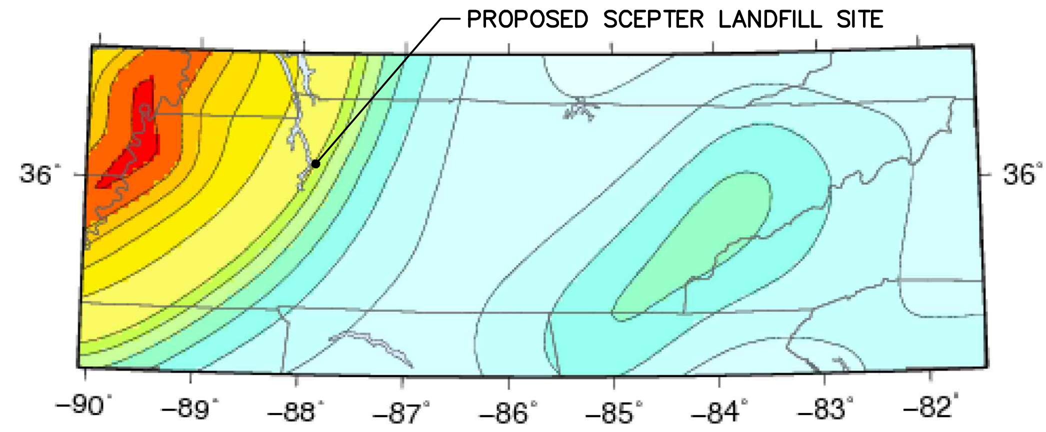

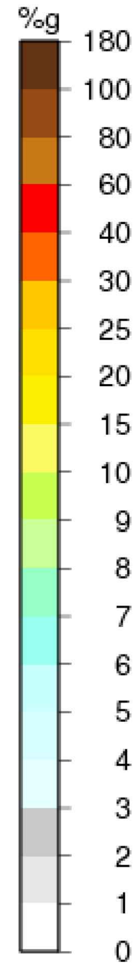

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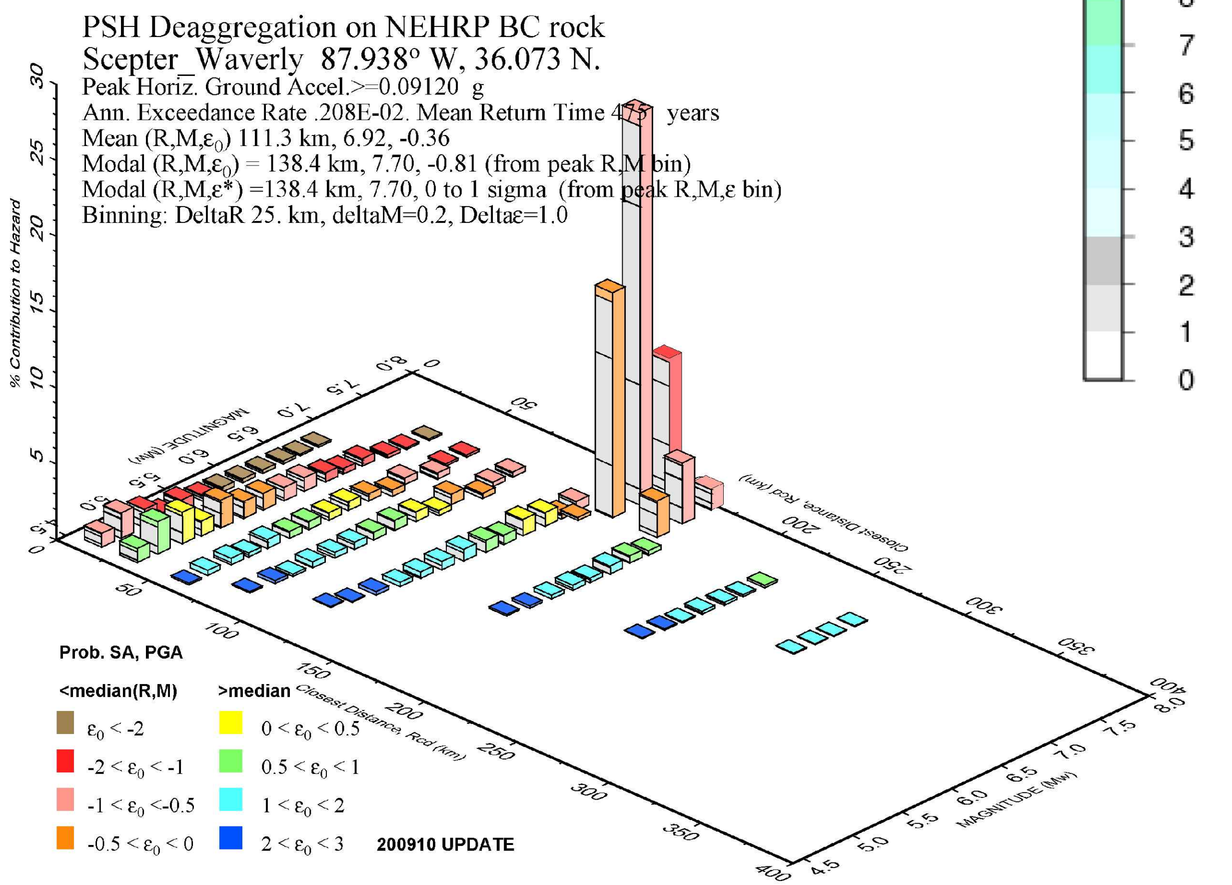

GMT 2016 Jul 5 13:21:48 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

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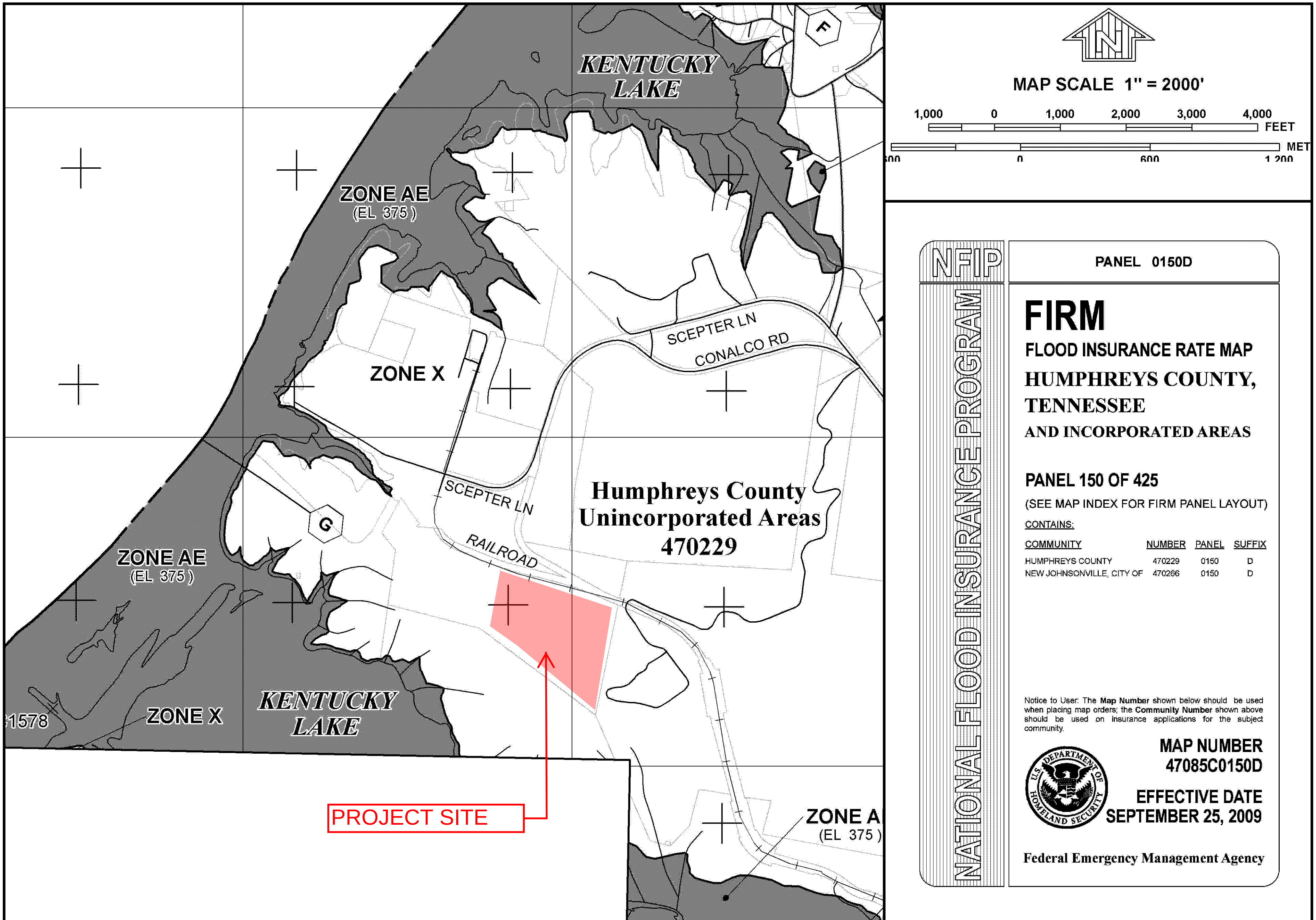
SCEPTER, INC.

WAVERLY, TENNESSEE

SEISMIC IMPACT ZONE 10% PROBABILITY OF EXCEEDANCE IN 50 YEARS

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	FIGURE
NP	MW	60398526	7/1/16	2



This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

FIGURE 3: FIRM MAP

PART B
CLOSURE/POST CLOSURE PLAN

SCEPTER, INC. – HUMPHREYS COUNTY, TENNESSEE

PART II PERMIT APPLICATION
SCEPTER, INC. DISPOSAL FACILITY – EAST PHASE SITE
CLOSURE AND POST-CLOSURE PLAN (REV. 2)

Prepared for
Scepter, Inc
Waverly, TN

January 11, 2017
April 5, 2018 Rev 2

Prepared by

AECOM

TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

This Closure and Post-Closure Plan (C/PC Plan) has been developed in support of the Part II Permit application for a Class II disposal facility to be located at Scepter, Inc. property in Humphrey's County, Tennessee. The disposal facility is to be located to the east of the existing landfill on land owned by Scepter. A new on-site solid waste disposal facility is required for the Scepter landfill to continue disposing of aluminum waste byproducts generated by the company. The proposed disposal facility is identified as the East Phase site, and is located immediately east of the existing landfill.

This Closure and Post-Closure Plan has been prepared in accordance with the Tennessee Department of Environment and Conservation's (TDEC's) Division of Solid Waste Management (TDSWM), Rule 0400-11-01-.03(2) and Rule 0400-11-01-.04(8).

The purpose of this document is to: (i) describe necessary activities associated with the closure of the disposal facility; and (ii) describe the monitoring and maintenance activities for the facility during the post-closure period. Information presented in this document is organized to be consistent with the permit application requirements of Rule 0400-11-01-.04(9). Sections within this application have been titled and enumerated corresponding to the regulations to aid in the review process.

1.1 SITE LOCATION AND DESCRIPTION

The Scepter, Inc. Industrial landfill is located in Humphreys County, Tennessee and lies between the cities of Waverly and New Johnsonville. The landfill can be accessed from Scepter Road north of Highway 70 traveling west from Waverly, Tennessee. The Scepter property occupies approximately 350 acres of land along the Tennessee River (Kentucky Lake). The East Phase landfill site encompasses approximately 31 acres within the proposed waste buffer boundary and varies in elevation from 420 to 620 feet msl.

1.2 EXPECTED YEAR OF CLOSURE

The facility is anticipated to receive 55,000 cubic yards of waste annually. Drawing P-9 depicts the final grades for the facility. Based on this design, there are approximately 3,000,000 cubic yards of disposal capacity available. The disposal facility is expected to provide 44 years of disposal capacity beginning in 2017. Therefore, completion of disposal operations and subsequent closure of the facility is expected in the year 2061. Table 1 provides a summary of the waste disposal capacity and anticipated life of each phase.

Table 1. Summary of Waste Disposal Capacity

Phase Development	Waste Placement Area (Acres)	Disposal Capacity (Cubic Yards)	Anticipated Life (Years)
Phase 1	7.49	284,836	4.4
Phase 2	8.66	836,104	12.8
Phase 3	7.23	1,096,353	16.8
Phase 4	3.02	635,304	9.7
Total	26.4	2,852,597	43.7

The final cover system may be constructed in phases as large areas of each cell are brought to final grade. Initially, the lower elevations of outer side slopes will receive a final cover, to be followed later by upper side slopes and finally the crest of the landfill. In this manner, the maximum area requiring final closure can be reduced.

1.3 Facility Contact Information

The following is a list of responsible parties involved in the permitting, design, operation, maintenance, quality control and quality assurance of the Scepter disposal facility.

Owner: Scepter, Inc.
Contact: Plant Manager
1485 Scepter Lane, Waverly, TN
Phone: 931-535-3565

As of the date of this manual, the plant manager is Mr. Chris Nicholas.

Solid waste correspondence should be directed to the responsible Landfill Coordinator. Presently the Landfill Coordinator is:

Contact: Brian Griffin, General Manager
1485 Scepter Lane, Waverly, TN
Phone: 931-535-3565

State contact information is as follows:

Contact: Tennessee Department of Conservation
Division of Solid Waste Management
Central Office
401 Church Street
5th Floor, L&C Tower
Nashville, TN 37243-1533

Phone: 615-532-0780
Fax: 615-532-0886

Contact as of the date of this manual is Mr. Patrick J. Flood, Director.

1.4 Document Format

Information presented in this document has been organized and presented consistent with the permit application requirements presented in Rule 0400-11-01-.03(2) and Rule 0400-11-01-.04(8). Sections within this document have been titled and enumerated consistent with the regulations to facilitate the review process. The following italicized text is copied verbatim from the aforementioned regulations. For clarity of discussion, each response is provided in bold text.

2.0 Closure Plan

2.1 Closure Requirements

Rule 0400-11-01-.04(8)(a)1. General Performance Standard - The operator must close the disposal facility or disposal facility parcel in a manner that:

(i) Minimizes the need for further maintenance; and

(ii) Controls, minimizes, or eliminates, to the extent necessary to prevent threats to public health and the environment, post-closure escape of solid waste, solid waste constituents, leachate, contaminated rainfall, or waste decomposition products to the ground or surface waters or to the atmosphere.

Sections 2.2 through 2.8 of this C/PC Plan outline the measures necessary to close the disposal facility in a manner that will minimize the need for further maintenance of the facility. It further specifies design requirements and procedures to control, minimize, or eliminate threats to public health and the environment. These measures include:

- Maintaining the leachate collection system
- Construction of a final cover system
- Surface grading to promote proper surface water management
- Implementing erosion and sediment control measures
- Maintaining the groundwater monitoring network

Rule 0400-11-01-.04(8)(a)2. The operator must care for a disposal facility or disposal facility parcel for the period of time after closure, specified in subparagraph (d) of this Rule, in a manner that assures that the performance objectives of part 1 of this subparagraph are continuously met.

Section 3.0 of this C/PC Plan outlines the measures necessary to care for the disposal facility during the post closure care period in a manner that meets the performance objectives of Rule 0400-11-01-.04(8)(a)1. These measures include:

- Inspection and maintenance of the leachate collection system
- Maintenance of the final cover system vegetative cover
- Maintenance of the erosion and sediment control measures
- Maintaining the groundwater monitoring network

Rule 0400-11-01-.04(8)(b). Adherence to Plan - The operator must initiate and complete closure activities and conduct post-closure care activities in accordance with the approved closure/post-closure care plan, if such plan has been prepared and approved for the disposal facility or disposal facility parcel being closed.

> Scepter will initiate and complete closure activities and conduct post-closure activities in accordance with the approved Closure/Post Closure Plan at the time of closure of the facility.

Rule 0400-11-01-.04(8)(c)1. Closure Requirements - The following requirements apply to active portions of the facility: The operator must notify the Division Director of his intent to close at least 60 days prior to the date he expects to begin final closure of the disposal facility or disposal facility parcel.

> Scepter will notify the Director of the Tennessee Division of Solid Waste Management of its intent at least 60 days prior to initiating closure activities.

Rule 0400-11-01-.04(8)(c)2. The operator must complete closure activities including grading and establishing vegetative cover in the shortest practicable time, not to exceed 180 days, after any fill areas or any portion of the fill areas have achieved final grade, unless the Commissioner allows otherwise in the permit. Permits may provide, or be modified to provide, minimum areas for closure which will be shown in closure plans. Such modifications of closure plans, for the sole purpose of identifying minimum closure areas, shall be deemed minor modifications. When these complete closure areas reach final grade, these areas shall be closed as otherwise provided in this part and within the 180 day time frame provided herein.

> Closure activities (including final cover placement, grading, drainage, and establishment of vegetative cover) of the facility will be completed within 180 days following the beginning of closure as previously mentioned, except where an extension is requested and approved by the Tennessee Division of Solid Waste Management.

2.2 FINAL COVER SYSTEM

Rule 0400-11-01-.04(8)(c)3. Unless otherwise noted in the permit a depth of compacted final cover material (e.g., soil) shall be placed on the disposal facility or disposal facility parcel in the shortest practicable time, not to exceed 90 days, after achieving final grade of any fill area or any portion of a fill area. At least the top twelve inches of this cover material shall be soil which will support the growth of suitable vegetation (e.g., topsoil).

(i) At Class I and Class II facilities the depth of final cover system shall be at least 36 inches of soil of which a minimum of 12 inches shall be for the support of vegetative cover.

The design of the final cover system shall be such that the infiltration volume of water will be equal to or less than the percolation volume through the bottom liner system or a design which includes a compacted soil layer of at least 24 inches which has a permeability no greater than 1 × 10-7 cm/sec, whichever is less. This design shall be supported by the use of the HELP model or other equivalent method approved by the Commissioner.

An alternate final cover system may be used provided that it is demonstrated to the satisfaction of the Commissioner that the final cover system provides equivalent or superior performance to the minimum performance standard in this subpart.

(ii) At Class III and Class IV facilities, unless the Commissioner determines that a greater depth is needed to achieve the general performance standard of subparagraph (a) of this paragraph, the depth of final cover shall be at least 30 inches of compacted soil. The final cover consists of an 18 inch low permeability layer overlain by a 12 inch protective layer.

(iii) At Class I, II, III, and IV facilities, with approval of the Commissioner any other low permeability layer construction techniques or materials may be used to provide the final cover, provided that it provides equivalent or superior performance to the requirements of this part.

Final cover will be placed over the disposal facility in the shortest time practical, after achieving final grade of any fill area or portion of a fill area.

An alternative final cover system is proposed for this facility. Details of the final cover system are illustrated on Drawing P-17. The proposed final cover system consists of (from top to bottom):

- A 24-inch thick vegetative cover soil/protective cover soil capable of sustaining native vegetation;
- A double-sided geocomposite drainage layer;
- A 40-mil linear low density polyethylene (LLDPE) flexible membrane liner barrier layer which has permeability much less than the required minimum 1×10^{-7} cm/sec.
- A geocomposite gas relief layer;
- A 6” barrier soil layer
- Intermediate soil cover.

The alternative final cover system will be installed upon a subgrade layer consisting of compacted waste material.

A final cover demonstration calculation is provided in Part F, Appendix F.3 of the permit application. The calculation demonstrates, through the use of the Hydraulic Evaluation of Landfill Performance (HELP) model, that the alternative final cover system provides equivalent or superior performance when compared to the final cover system prescribed by Rule 0400-11-01-.04(8)(c)3.

The final cover grading plan is presented on Drawing P-9. Drawings P-10, P-11, P-12 and P-13 provide details on the final grades expected as Phases 1, 2, 3, and 4 are developed. The final cover system may be constructed in stages as large areas are brought to final grade. Initially, outer side slopes may receive a final cover, to be followed later by the crest of the landfill.

The sequence of final cover construction will begin by constructing the 6 inch thick barrier cover soil. Much of the soil thickness may already be in place resulting from prior deployment of daily or intermediate cover soil. Thickness will be verified and compaction performed in a controlled manner. The geocomposite gas relief layer, a 40-mil thick textured Linear Low Density (LLDPE) geomembrane barrier layer and double-sided geocomposite drainage layer will be installed above the compacted barrier soil. The 24 inch thick protective cover soil that includes a top layer of vegetative soil will be installed above the geocomposite drainage layer and moderately compacted to aid in root development and prevent damage to the underlying geosynthetics. Finally, the surface of the cover will be seeded and vegetated.

Construction requirements of each final cover system component are provided in the Construction Quality Assurance (CQA) Plan located in Part D of the permit application.

As shown on Drawing P-14, the disposal facility will be graded to prevent the ponding of water. The grade of the final cover surface of the facility will not be less than 3 percent and no greater than 33.3 percent.

2.3 DRAINAGE SYSTEM

Rule 0400-11-01-.04(8)(c)4. The final surface of the disposal facility or disposal facility parcel shall be graded and/or provided with drainage facilities in a manner that:

(i) Minimizes precipitation run-on from adjacent areas onto the disposal facility or disposal facility parcel;

(ii) Minimizes erosion of cover material (e.g., no steep slopes);

(iii) Optimizes drainage of precipitation falling on the disposal facility or disposal facility parcel (e.g., prevent pooling); and

(iv) Provides a surface drainage system which is consistent with the surrounding area and in no way significantly adversely affects proper drainage from these adjacent lands.

The maximum design slope at the facility is a 3H:1V. The Storm Water Management Plan is shown on Drawing P-14. Erosion of soil material on the final cover system will be minimized through slope stabilization techniques, primarily consisting of proper vegetation. The maximum slope length of the steepest slope (between drainage benches) will be 95 feet.

The final cover grading plan which shows the surface water management system elements is presented on Drawing P-14. As shown on this drawing, the final cover will be graded to a minimum slope of 3 percent draining to surface water terraces with a minimum drainage slope of 1 percent. The cover system slopes promote precipitation run-off by maintaining positive grading to minimize the ponding of water.

Surface water terraces will intercept surface water runoff from the cover slopes and convey the runoff to letdown channels, which will convey the runoff to perimeter drainage ditches located at the toe of the cover system. Each perimeter drainage ditch is sloped to convey runoff to the sediment basin. Based on the natural topographic setting of the landfill, there is no surface water run-on from adjacent areas, thus exterior storm water diversion berms and channels are not necessary.

All drainage structures have been designed to accommodate at least a 25-year, 24-hour storm event with sufficient freeboard. The surface water management system has been designed to be consistent with the surrounding area and does not significantly affect proper drainage from or to adjacent lands.

2.4 VEGETATIVE COVER

Rule 0400-11-01-.04 (8)(c)5. In order to minimize soil erosion, as soon as practicable after final grading, the operator shall take steps as necessary to establish a protective vegetative cover of acceptable grasses over disturbed areas of the site. These steps shall include seeding, mulching, and any necessary fertilization at a minimum, and may include additional activities such as sodding of steeper slopes and drainage ways if such are necessary.

As soon as practicable after final grading, Scepter will take necessary steps to establish a protective vegetative cover of acceptable grasses over disturbed areas of the site. These steps shall include seeding, mulching, and any necessary fertilization at a minimum, and may include additional activities such as sodding or the use of reinforcement matting on steeper slopes and drainage ways if necessary. Application rates for seeding and fertilizing of indigenous grass/vegetation are provided in the Operations Manual included in Part A of this permit. Temporary erosion control blankets may be used if necessary to provide seedbed protection and prevent wash-out of seed and fertilizer during vegetation establishment.

2.5 Other Erosion and Sediment Control Measures

Rule 0400-11-01-.04 (8)(c)6. In addition to the drainage and grading requirements and vegetative cover requirements, the operator shall take other measures as may be necessary to minimize and control erosion and sedimentation (e.g., soil stabilization, sediment ponds) at the site.

In addition to the drainage, grading, and vegetative cover requirements, other measures such as soil stabilization through riprap protection, and sediment pond collection of surface water will be implemented to minimize and control erosion and sedimentation at the site. Letdown inlet areas, culvert outlet areas, and portions of the perimeter ditch will be lined with riprap. Additional erosion control problems will be addressed with appropriate structural and nonstructural sediment and erosion control practices as prescribed within the plans or the most recent edition of the Tennessee Erosion and Sediment Control Handbook.

2.6 Leachate Collection System

Rule 0400-11-01-.04 (8)(c)7. As required in this permit, or as otherwise necessary to prevent threats to human health and the environment, the operator shall establish and/or complete a system for collecting, removing, and treating leachate generated by the disposal facility or disposal facility parcel.

Leachate at the facility will be collected using a leachate collection and removal system depicted on Drawing P-6. Leachate collected from Phases 1, 2, 3, and 4 will be pumped through a force main to a storage tank system, where the leachate will be managed in accordance with Scepter's NPDES Permit #TN0002526.

2.7 Gas Collection System

Rule 0400-11-01-.04 (8)(c)8. As required in his permit, or as otherwise necessary to prevent threats to human health and the environment, the operator shall establish and/or complete a system for collecting and venting or otherwise controlling the vertical and horizontal escape of gases generated in the disposal facility or disposal facility parcel.

The Engineering plans show a gas migration control system that will be designed, constructed, operated, and maintained so any gases generated by decomposition or other reactions of the solid waste will be collected and vented to the atmosphere. The gas collection system will not allow a build-up of gas pressure that would compromise the final cover.

2.8 Borrow Area

Borrow areas used for construction of the facility and final cover system will be reclaimed by re-grading, stabilizing, and establishing permanent vegetation, within 180 days of ceasing borrow activities. Borrow and stockpile areas will be graded to allow positive drainage off-site to a maximum 3H:1V slope.

Additional erosion controls will be addressed with appropriate structural and non-structural sediment and erosion control practices as prescribed in the most recent edition of the Tennessee Erosion and Sediment Control Handbook.

2.9 CLOSURE CERTIFICATION AND NOTIFICATION

Rule 0400-11-01-.04 (8)(c)9. The operator must notify the Division Director in writing within 60 days of his completion of closure of the disposal facility or disposal facility parcel. Such notification must include a certification by the operator that the disposal facility or disposal facility parcel has been closed in accordance with the approved closure/post-closure care plan. Within 21 days of the receipt of such notice the Division Director shall inspect the facility to verify that closure has been completed and in accordance with the approved plan. Within 10 days of such verification, the Commissioner shall approve the closure in writing to the operator. Closure shall not be considered final and complete until such approval has been made.

Scepter will notify the Division Director in writing within 60 days of completion of closure of the disposal facility. The notification will include a certification by Scepter that the disposal facility has been closed in accordance with this C/PC Plan.

3.0 POST-CLOSURE PLAN

3.1 POST-CLOSURE CARE PERIOD

Rule 0400-11-01-.04(8)(d) Post-Closure Care Period - For Class I and Class II disposal facilities, post-closure care must continue for 30 years after the date of final completion of closure of the disposal facility or disposal facility or parcel unless a shorter period is established in the approved closure/post-closure care plan. For Class III and IV disposal facilities, post-closure care must continue for 2 years after the date of final completion of closure of the facility or facility parcel. The post-closure care period may be reduced or extended based on cause by amendment of the approved closure/post-closure care plan as provided in rule 0400-11-01-.03(2)(c).

Scepter will provide post-closure care for 30 years after the date of final completion of closure of the disposal facility unless this C/PC Plan is amended to provide for a shorter period.

3.2 POST-CLOSURE CARE ACTIVITIES

3.2.1 FINAL CONTOURS AND DRAINAGE SYSTEM

Rule 0400-11-01-.04(8)(e)1. Post-Closure Care Activities - During the post-closure care period, the operator must, at a minimum, perform the following activities on closed portions of his facility: Maintain the approved final contours and drainage system of the site such that the objectives of part (c) 4 of this paragraph are continuously met;

The approved final cover and drainage system will be maintained at the site. The effectiveness of the final cover will be maintained by making repairs to the cover as necessary to correct the effects of subsidence and erosion, as well as preventing run-on and run-off from eroding/damaging the final cover system. If settlement or other structural problems occur in the final cover system, the cover will be re-graded. The problem area will be stripped of the vegetation layer and fill dirt will be applied to the area. The disturbed area will be covered with soil and reseeded as specified in the design. If excessive surface cracks appear on the soil cover, the cracks will be properly graded with suitable soil and appropriate vegetative cover will be re-established to prevent the infiltration of surface water.

Accumulated sediment will be removed from sediment basins when the sediment level impedes on the wet weather storage volume. The sediment basins will be drained and excavated using a front-end loader, dozer, or other equipment for cleaning. The outlet and inlet structures of the sediment basins will be maintained by the operator throughout the life of the disposal facility and during the post-closure period.

3.2.2 VEGETATIVE COVER

Rule 0400-11-01-.04(8)(e)2. Ensure that a healthy vegetative cover is established and maintained over the site.

The vegetative cover will be inspected regularly to maintain a healthy stand of vegetation. Areas containing distressed vegetation will be reseeded. The vegetative cover over the site will be maintained by mowing on a regular schedule. Initially the grass will be cut quarterly; however, once the grass is established, it will be cut twice a year or more frequently as needed. The mowing schedule is intended to limit the growth of weeds or rooting of brush species that could undermine the final cover. If an area has less than 75 percent coverage by grass, the area will be reworked and reseeded. Fertilizer may be applied to promote the re-establishment of a self-sustaining vegetative cover. Significant depressions or gullies that develop will be promptly repaired by filling with soil and seeding. The details of the vegetative cover are provided in the CQA plan.

Rule 0400-11-01-.04(8)(e)3. Maintain the drainage facilities, sediment ponds, and other erosion/sedimentation control measures (if such are present at the landfill), at least until the vegetative cover is established sufficiently enough to render such maintenance unnecessary.

Until vegetation of the final cover is fully established, sediment transport will be retarded by temporary silt fences. Sediment transported from the cover before vegetation is fully established will be conveyed to one of the sediment basins. Should excessive cleaning and maintenance of the sediment basin be needed due to erosion of soil from the cover, temporary sediment control measures will be installed to reduce the sediment load until the vegetative cover is fully established. Storm water channels will be lined with grass or riprap to prevent erosion. The channels will be inspected regularly and after major storm events for structural and erosion problems. If damage to a channel is discovered, it will be repaired as appropriate.

3.2.3 Leachate Collection System

Rule 0400-11-01-.04(8)(e)4. Maintain and monitor the leachate collection, removal, and treatment system (if such is present at the facility);

Maintenance and monitoring of the leachate collection system will include inspecting the leachate collection pump systems and side slope risers, removing sediment and debris (if needed), verifying operation of pumps, and ensuring that outlets are clear and unobstructed.

Leachate collection will continue during the post-closure period.

3.2.4 Gas Collection System

Rule 0400-11-01-.04(8)(e)5. Maintain and monitor the gas collection and control system (if such is present at the facility);

Maintenance and monitoring of the gas collection system will include inspecting the passive vents and ensuring that the pipes are undamaged and outlets are clear and unobstructed.

3.2.5 Groundwater Monitoring Plan

Rule 0400-11-01-.04(8)(e)6. Maintain and monitor the ground and/or surface water monitoring system (if such is present at the facility). The monitoring system and sampling and analysis program established in the permit shall be continued during the post-closure care period, unless the permit is modified to establish a different system or program. Monitoring data must be reported in writing to the Division Director within 30 days after the completion of the analyses.

Groundwater monitoring during the post-closure care period will be performed in conformance with the Groundwater Monitoring Plan presented in Part C of the permit application. Monitoring data will be reported in writing to the Division Director within 30 days after the completion of the analyses.

3.2.6 Inspections

To comply with Rules 0400-11-01-.04(8)(e)1-6 (Sections 3.2.1 through 3.2.5), a visual inspection of the site will be made on a regular basis for the duration of the post-closure care period. Inspections will be completed on a quarterly basis for the initial year following closure and on an annual basis thereafter. Maintenance or other corrective measures needed to prevent the deterioration of the closure system will be identified during the inspections. A post closure checklist that will be used for inspections to identify needed maintenance is provided as Appendix B.2 of this Closure and Post-Closure Plan.

Features to be inspected include active as well as closed portions of the disposal site, the leachate collection system, surface-water and ground-water monitoring locations, security devices, vegetative cover, confirming proper grades for positive drainage, sediment ponds, and other erosion/sedimentation control features. Each inspection will be documented and will include, at a minimum, the following information: date and time of inspection, name of inspector, notation of observations made, nature of any remedial actions to be taken, and recommendation for corrective measures.

POST CLOSURE MAINTENANCE AND INSPECTION SCHEDULE	
ACTIVITY	FREQUENCY
Visual Site Inspection	1st Year: Quarterly 2nd-30th Year: Annually
Leachate Collection System	
Surface-water	
Ground-water Monitoring Locations	
Security devices	
Vegetative Cover	
Final Grading/Drainage	
Sediment Basin	
Erosion/Sediment Control Features	
Vegetation Maintenance (Grass Cutting)	1st Year: Quarterly 2nd-30th Year: Twice Annually
Sediment Basin Cleanout	½ Wet Pool Compromised Compromised Forebay

3.2.7 POST-CLOSURE CERTIFICATION

Rule 0400-11-01-.04(8)(e)7. Following completion of the post-closure care period for each SWLF unit, the owner or operator must file with the Department a certification verifying that post-closure has been completed in accordance with the post-closure plan.

Following completion of the post-closure care period for the disposal facility, Scepter will file with the Department a certification verifying that post-closure has been completed in accordance with the C/PC plan.

3.3 POST-CLOSURE USE

Rule 0400-11-01-.03(2)(b)2(v). A description of the planned uses of the property during the post-closure care period.

Following development and closure of the landfill facility, the property is expected to be used as an open space. Any other uses will first be approved by the appropriate regulatory agencies.

4.0 NOTICE IN DEED TO PROPERTY

Rule 0400-11-01-.04(8)(f) Notice in Deed to Property - The operator must ensure that, within 90 days of completion of final closure of the facility and prior to sale or lease of the property on which the facility is located, there is recorded, in accordance with State law, a notation on the deed of property or on some other instrument which is normally examined during a title search that will in perpetuity notify any person conducting a title search that the land has been used as a disposal facility and its use is restricted in accordance with the approved closure/post-closure plan.

> Within 90 days of completion of final closure activities of the facility and prior to final sale or lease of the property on which the facility is located, Scepter will ensure that there is recorded, in accordance with State law, a notation on the deed to the property or some other instrument, which is normally examined during a title search that will in perpetuity notify any person conducting a title search that the land has been used as a disposal facility, and the use of the property is restricted in accordance with the approved closure/post-closure plan.

Rule 0400-11-01-.04(8)(g) Open Dump or Unauthorized Dump Closing - A person discontinuing the use of an open dump or unauthorized dump, whether on his own initiative or at the direction of the Commissioner, shall take all the actions determined by the Commissioner to be necessary, including but not limited to, the following: [remainder omitted]

> Because there is neither an Open Dump nor Unauthorized Dump on site; Rule 0400-11-01-.04(8)(g) is not applicable to this facility.

5.0 AMENDMENT OF PLAN

Rule 0400-11-01-.03(2)(c). Amendment of Plan – The approved closure/post-closure care plan may be amended at any time during the active life of the facility or during the post-closure care period as set forth in this subparagraph.

1. The operator may request to amend the plan to alter the closure requirements, to alter the post-closure care requirements, or to extend or reduce the post-closure care period based on cause. The request must include evidence demonstrating to the satisfaction of the Commissioner that:
   (i) The nature of the facility makes the closure or post-closure care requirement(s) unnecessary; or
   (ii) The nature of the facility supports reduction of the post-closure care period; or
   (iii) The requested extension in the post-closure care period or alteration of closure or post-closure care requirements is necessary to prevent threats to human health and the environment.
2. Such plan amendments shall be processed as modifications to the permit. However, the Commissioner may decide to modify the plan if he deems it necessary to prevent threats

to human health and the environment. He may extend or reduce the post-closure care period based on cause or alter the closure or post-closure care requirements based on cause. However, no such modifications shall be initiated until the operator has been notified of such proposed action and provided the opportunity to be heard on the matter.

Amendments to the plan as requested by Scepter will include the necessary information required by rule.

APPENDIX B.1

Cost Estimate for Closure and Post-Closure

COST PER ACRE FOR FINAL COVER

A. Location of Soil Materials

The barrier layer (waste cover) soil, protective cover material and topsoil necessary for final cover construction of the synthetic cap area will be obtained either from on site stockpiles, from excavation for cell construction or drainage channel construction, or from borrowed areas on site, or off-site, as needed.

B. Cost per acre for acquisition, placement and compaction of barrier layer soil.

(1) Acquisition of barrier layer soil

(a) Quantity of material needed per acre (yd^3/acre):	807
(b) Excavation unit cost ($/$yd^3$) (obtained on site)	$2.00
(c) Acquisition cost ($/acre)	$1614.00

(2) Placement and Compaction

(a) Placement/spreading unit cost ($/$yd^3$)	$2.00
(b) Compaction cost ($/$yd^3$)	$1.10
(c) Placement and compaction cost ($/acre)	$2,502.00

(3) Acquisition, placement and compaction cost ($/acre) (Line (1)(c) + (2)(c))	$4,116.00

C. Cost per acre for acquisition, placement and compaction of cap cover layer and topsoil including testing.

(1) Acquisition of cap cover material

(a) Quantity of material needed per acre (yd^3/acre):	2,420
(b) Excavation unit cost ($/$yd^3$) (obtained on site)	$2.00
(c) Acquisition cost ($/acre)	$4840.00

(2) Acquisition of topsoil

(a) Quantity of topsoil needed per acre (yd^3/acre):	807
(b) Excavation unit cost ($/$yd^3$):	$5.00
(c) Acquisition cost ($/acre)	$4035.00

(3) Placement and compaction/cap cover

(a) Placement/spreading unit cost ($/$yd^3$)	$2.00
(b) Compaction cost ($/$yd^3$)	$1.10
(c) Placement and compaction cost ($/acre)	$7502.00

(4) Placement/topsoil

(a) Spreading unit cost ($/$yd^3$):	$2.75
(b) Placement cost ($/acre):	$2,219

(5) Testing

(a) Soil classification ($/acre)	$50
(b) Survey control for cover thickness and proper slopes ($/acre)	$225
(c) Density testing ($/acre)	$350
(d) Testing cost ($/acre)	$625

(6) Acquisition, placement and compaction cost ($/acre) (Line (1)(c) + (2)(c) + (3)(c) + (4)(b) + (5)(d))	$19,221

D. Cost per acre for installation of synthetic liner system

(1) Geocomposite Gas Relief Layer and Vents ($/acre)	$27,500
(2) Flexible Membrane Liner -40 Mil LLDPE ($/acre)	$27,500
(3) Geocomposite Drainage Layer ($/acre)	$32,500
(4) Synthetic Liner System Cost ($/acre) (Line (1) + (2) + (3))	$87,500

E. Cost per Acre to Establish Vegetation

(1) Seeding, Fertilization, Mulching unit cost ($/acre)	$ 2,000

F. Cost per Acre to Certify Closure

Registered Professional Engineer -

(1) Initial review of closure plan (hrs.)	8
(2) Total number of inspections (3 per phase)	12
(3) Inspection time required (hrs./visit)	8
(4) Total inspection time (hrs.) (Line (F)(2) * (F)(3))	96
(5) Prepare final documentation (20 hrs. per phase)	80
(6) Total engineer time (hrs.): (Line (F)(1)+(F)(4)+(F)(5))	184
(7) Engineer unit labor cost ($/hr.):	$125
(8) Professional engineer cost ($):(Line (F)(6) * (F)(7))	$23,000
(9) Area of site permitted for Synthetic Cap (acres):	26.4
(10) Closure certification cost ($/acre): ((F)(8)/(F)9))	$871

G. Total Cost per Acre for Final Cover — $113,708

B(3) + C(6) + D(4) + E(1) + F(10)

VI. OTHER CLOSURE COSTS

A. Property Deed Notation Costs are estimated at $2,500.

B. Drainage and Erosion Control Costs
The work required for closure of the site in addition to the placement of final cover includes the drainage and erosion control structures. The costs for drainage channels and erosion control devices are approximately $120,000 (based on $60/LF).

C. Survey Costs
Surveys will be conducted to verify thicknesses and elevations of various layers as described above. The cost for survey work is approximately $92,000 (based on $3,500/Acre).

E. Other Closure Costs Total – ((A)+(B)+(C)) $215,000

VII. CLOSURE COST ESTIMATE
26.4 Ac. x $113,708/Ac. $3,001,891
Add VI.E: $215,000

Total Closure Cost Estimate $3,217,000

VIII. PARTIAL CLOSURE COST ESTIMATE

Partial Closure Cost Estimate			
Phase	Closure Area (ac)	Unit Price ($/ac)	Cost per Phase
3	8.8	113,708	$1,000,630
4	4.1		$466,203
Final	13.5		$1,535,058

POST-CLOSURE COST ESTIMATE - 30 years

A. Cost for Semi-Annual Inspections and Reports

1. Inspection

(a.) Number of inspections during post-closure period (semiannual inspections for 30 years.)			62
(b.) Inspector time required (hrs./inspection)			2
(c.) Inspector unit labor cost ($/hr.)			$80
(d.) Inspection cost ($) (Line Al(a) x Al(b) x Al(c))		$9,920	

2. Report Preparation

(a.) Number of reports during post-closure period	62
(b.) Cost per report ($)	$200
(c.) Report cost ($) – (Line A2(a) x A2(b))	$12,400

3. Inspection and Report Cost ($) (Line A1(d)+A2(c)) $22,320

B. Cost for Maintenance of Final Cover and Vegetation

The cost for cover maintenance and vegetation shall be twenty (20) percent of the cost per acre calculated for protective cover & topsoil and vegetation in the closure plan.

1. Final Cover Maintenance

a. 20% of the cost of placement of protective cover & topsoil and vegetation (as determined by Items C(6) and E (1) of the Closure Plan = $21,221/acre):	$4,245
b. Total area of site permitted for filling (acres)	26.4
c. Total area of site for final cover (acres)	26.4
d. Cover Maintenance Cost ($) Line B1 (a) x B1 (c)	$112,068

C. Cost for Vegetation Control

1. Mowing

a. Mowing frequency (visits/30 years)	62
b. Area to be mowed (acres/visit)	26.4
c. Mowing unit cost ($/acre)	$200
d. Vegetation Control Cost ($) Line Cl(a) x Cl(b) x Cl(c)	$327,400

D. Cost for Maintenance of Access Control and Benchmarks

1. Access Control Maintenance $1000

2. Benchmark Maintenance Cost (if any) ($) $500

3. Access Control and Benchmark Repair Cost ($)

   (Line Dl + D2) $1,500

E. Cost for Leachate Collection System Monitoring and Maintenance

1. Leachate Collection System Inspection

   a. Inspection frequency (inspection/30 years) 62

   b. Inspection time required (hrs./inspection) 2

   c. Inspector unit labor cost ($/hr.) $80

   d. Inspection cost ($) (Line El (a) x El (b) x El (c)) $9,920

2. Leachate Collection System Maintenance

   a. Line Cleaning $10,000

   b. Storage tanks/valve maintenance $5,000

   c. Pump Repair $10,000

   d. Maintenance cost ($) (Line E2(a) + E2(b)+E2(c)) $25,000

3. Electricity Costs ($500/Year) $15,000

4. Leachate Collection Monitoring and Maintenance Cost ($)

   (Line E1(d) + E2(c) + E3) $50,000

F. Cost for Methane Control System Monitoring and Maintenance

1. Methane Control System Monitoring

   a. Gas monitoring frequency (visits/10 years) N/A

   b. Time required to monitor (hrs./visit) N/A

   c. Contract lab technician unit labor cost ($/hr.) N/A

d. Gas monitoring cost ($)

LineFl(a)xFl(b)xFl(c) N/A

2. Gas Monitoring Well Maintenance

a. Maintenance frequency (visits/30 years) N/A

b. Monitoring wells needing maintenance per visit N/A

c. Maintenance time required (hrs./well) N/A

d. Unit labor cost ($/hr.) N/A

e. Gas Monitoring well maintenance costs ($)

(Line F2(a) x F2(b) x F2(c) x F2(d)) N/A

3. Gas Monitoring and Maintenance Cost ($)

(Line F1 (d) + F2(e)) N/A

G. Cost for Groundwater Monitoring System Maintenance

1. Monitoring Well Maintenance

a. Maintenance frequency (visits/30 years) 30

b. Number of monitoring wells needing maintenance per visit 1

c. Maintenance time required (hrs./well) 4

d. Unit labor cost ($/hr.) $80

e. Monitoring well maintenance costs ($)

(Line Gl(a) x Gl(b)xGl(c)xGl(d)) $9,600

2. Monitoring Well and Parts Replacement

a. Number of wells needing replacement during post-closure period 2

b. Existing monitoring well sealing unit cost ($/well) $1500

c. New monitoring well construction unit cost ($/well) $3500

d. Monitoring well replacement cost ($) (Line G2a x (G2(b) + G2(c))) $10,000

e. Number of pumps needing replacing during post-closure period N/A

f. Pump cost ($) - ($/pump) ` N/A

g. Pump cost ($) - Line G2(e) x G2(f) N/A

3. Groundwater monitoring system maintenance cost

Line Gl(e) + G2(d) + G2(g) $19,600

H. Cost for Groundwater Monitoring

1. Groundwater Monitoring

a. Number of required monitoring wells 3

b. Monitoring frequency (semiannual sampling for 30 years) 60

c. Sampling and analysis cost ($/well) $1,250

d. Groundwater Monitoring Cost ($)

(Line la x Line 1b x Line 1c) $225,000

I. Cost for Leachate Hauling

1. Leachate Pumping and Hauling

a. Leachate removal frequency (loads/30 years) 500

b. Quantity to be managed (gals/load) 3,000

c. Truck capacity (gallons) 3,000

d. Number of loads/visit (Line Il(b) / Il(c)) 1

e. Pumping and transportation unit cost ($/load) $20

f. Leachate Hauling Cost ($) (Line I1(a) x I1(d) x I1(e)) $10,000

J. Cost for Leachate Disposal

1. Leachate Treatment

a. Leachate Disposal Frequency (years) 30

b. Volume of Leachate requiring disposal (gals.) 1,560,000

c. Disposal unit cost (Annual Lump Sum) $0.00

d. Leachate Disposal Cost ($) (Line J1 (a) x J1 (c) $0.00

K. Total Post-Closure Cost

Total of (Line A.3. +B.1.d.++C.1.d.+D.3.+E.4.+G.3.+H.1.d.+I.1.F.+J.1.d. $767,888

APPENDIX B.2

Post-Closure Care Checklist

Post-Closure Inspection and Maintenance Report Form

Location/Feature Inspected	Date of Inspection	Time of Inspection	Inspection after significant rainfall	Observed Maintenance Deficiency (see list below)	Condition (see below)	Corrective Action Required / Other Remarks	Date Repair Completed
Example: Top of Cover, west side							

Condition Code: G = Good M = Marginal, needs maintenance within 7-days P = Poor, needs immediate maintenance C = Needs to be cleaned O = Other (Explain in Corrective Action Section)

INSPECTION CHECKLIST			
GROUND SUBSIDENCE	SLOPE STABILITY	MONITORING WELLS	OUTFALLS/HYDRAULIC STRUCTURES
1. Evidence of cracking	9. Rotational/block failure	15. Well identification not visible	22. Erosion observed
2. Evidence of depression	10. Maintenance sloughing	16. Well cap unlocked or insecure	23. Sedimentation observed
3. Evidence of sinkhole	11. Evidence of seeps	17. Ponded water in well vicinity	24. Inlet/outlet obstruction
4. Evidence of ponding	CHANNELS/LININGS	18. Subsidence in well vicinity	GENERAL/OTHER SITE FEATURES
FINAL COVER	12. Erosion observed	19. Erosion in well vicinity	25. Evidence of unauthorized entry
5. Animal burrows observed	13. Sedimentation observed	20. Collision damage	26. Damage/missing facility signage
6. Stressed vegetation observed	14. Lining deterioration/displacement	21. Well casing degradation	27. Other deficiency observed
7. Undesired vegetation present			
8. Excavation in final cover			

PRECIPITATION DATA		
Date	Rain Data	
	Ins.	Amount

Inspector Information:			
	Name	Title	Signature

PART C
GROUNDWATER MONITORING PLAN

SCEPTER, INC. – HUMPHREYS COUNTY, TENNESSEE

GROUND WATER DETECTION MONITORING PLAN
SCEPTER, INC. DISPOSAL FACILITY

Prepared for

Scepter, Inc
Waverly, TN

July 1, 2016

Prepared by

AECOM

TABLE OF CONTENTS

Figures

Appendices

1.0 Introduction

1.1 Background

This document provides field procedures and sampling protocols, analysis, and record keeping requirements that are applicable to the Scepter, Inc. Industrial Landfill in Waverly, Tennessee (IDL 43-102-0078) as provided in Tennessee Department of Environment and Conservation (TDEC) Rule 0400-11-01 for the groundwater monitoring of a solid waste disposal facility.

A new on-site solid waste disposal facility is required for the Scepter landfill to continue receiving aluminum waste byproducts generated by the company. The new area of the disposal facility is identified as the East Phase site, and is located immediately east of the existing West Phase Site. The Groundwater Monitoring Plan (GWMP) describes procedures and protocols to evaluate the potential impacts from both the East and West Phases of the disposal facility on groundwater quality beneath the site. Figure 1 shows the location of the property on a regional topographic map.

The procedures to be adhered to and the sample analyses to be performed during groundwater detection monitoring activities will be completed consistent with the protocols identified herein.

1.2 Certification

I certify under penalty of law that this document was prepared by me or under my supervision in accordance with a system designed to assure that qualified personnel properly gathered and evaluated the information submitted. The information submitted is, to the best of my knowledge and belief, true, accurate, and complete.

Craig A. Bernhoft, P.G.
June 29, 2016

2.0 FACILITY SETTING

2.1 LOCATION AND DESCRIPTION

The Scepter, Inc. Industrial landfill is located in Humphreys County, Tennessee and lies between the cities of Waverly and New Johnsonville. The landfill can be accessed from Scepter Road, north of Highway 70 traveling west from Waverly, Tennessee. The Scepter property occupies approximately 350 acres of land along the Tennessee River (Kentucky Lake). The area encompassed by the disposal facility (East and West Phases) is approximately 70 acres and varies in elevation from 420 to 620 feet msl.

2.2 MONITORING WELL LOCATIONS

The groundwater monitoring system is designed to assess the quality of groundwater unaffected by disposal processes and to monitor groundwater quality at the downgradient compliance boundary of the disposal facility. The groundwater monitoring system consists of two upgradient (background) wells and several downgradient monitoring wells. The locations of the monitoring wells are shown on Figure 2. The active West Phase is currently being monitored by the wells shown in the table below. The East Phase will be monitored by the additional East Phase monitoring wells that were installed during the East Phase hydroleologic groundwater investigation. The additional hyrdogeologic investigation drilling activities took place from December 7, 2015 to December 17, 2015. The drilling locations were initially selected to characterize the uppermost aquifer and evaluate subsurface conditions to form the basis of design. The wells were installed following the procedures established by the USEPA SESDGUID-101-R0, Design and Installation of Monitoring Wells. The completed wells were fitted with a protective steel casing, a concrete well pad and steel bollards cemented in place for protection. The monitoring wells at the Scepter facility were surveyed to establish vertical and horizontal positioning that is used for determining groundwater elevations during sampling events.

During the semi-annual groundwater monitoring event, groundwater samples will be collected from some or all of the following monitoring wells depending on disposal activities.

West Phase	East Phase
Background Well – MW-1A	Background Well – MW-E1
Down-gradient Well – MW-2A	Downgradient Well – MW-E2
Down-gradient Well – MW-3	
Down-gradient Well – MW-4	

3.0 Groundwater Monitoring Program

This section provides a discussion of the Groundwater Monitoring Program for the landfill in accordance with the TDEC Solid Waste regulations established in Rule 0400-11-01-.04.

3.1 Detection Monitoring Program

Groundwater samples are initially being collected and analyzed from the East Phase site monitoring wells for the indicator parameters on a quarterly schedule during the first year following installation and are scheduled for semi-annual sampling following the startup of landfill operations. The purpose of this pre-operational (prior to placement of waste) sampling for the East Phase is to establish statistical baseline concentrations for analytes in each well. During the sampling events, groundwater samples from the monitoring wells will be analyzed for the approved parameters that are required by TDEC Division of Solid Waste Management (DSWM) to be sampled semi-annually at the Scepter landfill, as listed below:

Aluminum	Arsenic	Barium	Cadmium	Calcium	Chromium
Cobalt	Copper	Iron	Magnesium	Manganese	Nickel
Potassium	Silver	Sodium	Vanadium	Zinc	Chloride
Nitrate	Sulfate	Ammonia - Nitrogen		Total Phosphorus	
COD	TOC	TDS			

The analytical methods used to analyze all samples will be the appropriate methods from EPA Publication SW-846. The particular SW-846 method used will have laboratory reporting limits of the lowest practical quantitation limits (PQL) (i.e., laboratory reporting limits) that can be reliably achieved within specified limits of precision and accuracy. The laboratory reporting limits (PQL or PQL equivalent such as EQL, RL, LOQ, etc.) will be the lowest practical quantitation limits that can be reliably achieved within specified limits of precision and accuracy, with a target of at least four times below all established groundwater protection standards in Appendix I of Rule 0400-11-01-.04 or other groundwater protection standards approved by the DSWM.

Upon completion of the quarterly events conducted in the first year, the concentrations of the groundwater constituents will be evaluated to determine if they are representative background concentrations. Upon approval by TDEC, these background concentrations will be used to screen all subsequent groundwater samples collected during the Detection Monitoring Program.

3.2 Data Quality Objectives

Data Quality Objectives (DQOs) are determined by the intended use of the analytical data. The goal of this project is to utilize the existing groundwater monitoring network for detection monitoring for the parameters as required and permitted by the TDEC DSWM.

The main purpose of this plan is to develop and ensure that the implementation of procedures to be used for field sampling, chain-of-custody, laboratory analysis, and chemical data reporting will provide legally defensible data of known quality. The quality of the data generated must support the end use of the data. The minimum data documentation requirements for the groundwater monitoring activities are outlined as follows:

Sampling Quality Control Data and Information:

- Chain-of-Custody
- Date and time each sample was taken
- Map or diagram indicating sample locations
- Any notable observations (color, clarity, texture, reaction with preservatives, etc.)
- Trip Blank
- Equipment blank (rinsate blank)
- Identity of field duplicates (a minimum of one duplicate for every 20 or fewer samples)
- Sample for site specific MS/MSD

Laboratory Quality Control Data and Information:

- Completed Chain-of Custody
- Date and time of receipt at the laboratory
- Condition of samples upon receipt at the laboratory
- Sample identification number or designation
- Sample preparation, extraction, cleanup, or digestion method(s) and date(s)
- Analytical method (name, number, and source) and date of analysis
- Final analytical results
- Case narrative (Includes deviations from standard analytical or preparatory procedure(s); quality control problems encountered—whether stemming from system, instrumentation, analyst error, or sample matrix; corrective measures taken; if corrective measures as called for in the method were not taken; results of corrective measures taken; etc.)

3.3 REPORTING

Within sixty (60) days following the last day of the sampling event, Scepter will submit to DSWM the groundwater sampling and analysis results, statistical determinations, and associated recordings of the groundwater surface elevations. The groundwater monitoring reports will provide the following items as specified in TDEC's Ground Water Monitoring Guidance for Solid Waste Landfill Units (GW Monitoring Guidance Document):

1. Description of sampling procedures, field measurements (i.e., water quality parameters), purge volumes, dates and times of sampling, and weather conditions;
2. Elevation (MSL-mean sea level) of each monitoring well's top of casing and groundwater surface;
3. Groundwater flow direction and rate across the site;
4. A description of the results of the inspections of all monitoring wells pad, above-ground casing, locking cap, and lock;
5. A scaled map of the facility showing the locations of all monitoring points and the MSL potentiometric surface determined from water level measurements collected during the event, the property boundaries, and active and closed fill areas;
6. Summary of sampling parameters and analysis methods;
7. Copies of chain of custody forms and laboratory reports;
8. Tabulated sample results compared against background groundwater quality concentrations and groundwater protection standards;
9. Summary of the statistical method used for determining a statistically significant increase meeting the requirements of Rule 0400-11-01-.04 and the results of the statistical analysis;
10. Scaled site base map that presents the location of each monitoring well and concentrations of the constituents/parameters that were found to statistically exceed background concentrations or groundwater quality protection standards;
11. A schedule for the next sampling event; and,
12. A certification meeting the requirements of Rule 0400-11-01-.02(3)(a) 7, 8, and 10.

3.4 STATISTICAL DATA EVALUATION

Consistent with the current monitoring program, an interwell statistical analysis, consistent with both the requirements of Rule 0400-11-01-.04 and the statistical characteristics of the historical groundwater monitoring data will be the methodology applied to future monitoring data to assure timely detection of statistical exceedances at the compliance monitoring boundary. Normality testing will be used to evaluate the data at a 0.01 or 0.05 significance level, in accordance with Rule 0400-11-01-.04 (7) (a) 4. (vi). A high percentage of non-normally distributed data is generally due to a high percentage of measurements below analytical reporting limits.

When the statistical characteristics of prior monitoring data match the distribution described above, that indicates that the prediction interval methods adapted by Gibbons (1990 and 1994) would likely be applicable to the groundwater detection monitoring program. Either parametric or nonparametric prediction interval methods would be applied depending on normality of individual constituent data. In general, one-sided upper prediction limits (UPLs) derived from n baseline measurements from each well and having a (1-α) probability of including at least one of two future measurements at the well would be computed for each constituent using the methods of Gibbons (1994, pp. 8-76), where α is the Type 1 (false-positive) error level. Future sample measurements from each well will be compared to baseline UPLs developed from a minimum of four baseline monitoring events for each well. Once waste is placed in the landfill, the background or baseline dataset is complete and statistics can be computed. If a new measurement exceeds a UPL, one verification resample would be collected from each monitoring well having a statistical exception. The resample would be analyzed only for the exceeded constituent(s). Should the resample result exceed the UPL, the exception would be deemed statistically significant; otherwise, the original UPL exception would be considered insignificant. Resampling may also be unnecessary and therefore waived if samples are chronically above the UPL for a given metal and results are within the historical range of data observed at this well.

Tolerance intervals will be constructed using ProUCL, a statistical analysis program developed by the United States Environmental Protection Agency (USEPA) or similar data management and software programs. The statistical analysis includes characterization of the data, assumption testing, specific statistical comparisons to test hypotheses, and interpretation of results. All methods used will be in accordance with the following USEPA guidance and ASTM standards:

- "Statistical Analysis of Ground-Water Monitoring Data at RCRA Facilities – Unified Guidance" Office of Resource Conservation and Recovery Program Implementation and Information Division USEPA, EPA 530-R-09-007, March 2009.
- ASTM D7048 Standard Guide for Applying Statistical Methods for Assessment and Corrective Action Environmental Monitoring Programs.
- ASTM D6312 Standard Guide for Developing Appropriate Statistical Approaches for Groundwater Detection Monitoring Programs.

Monitoring well data will also be compared to the TDEC MCLs listed in Appendix III of Rule 0400-11-01-.04.

3.5 Assessment Monitoring

If groundwater detection monitoring results indicate either a statistically significant increase above background for any naturally occurring constituent or a confirmed detection of any required monitoring parameter that does not occur naturally, TDEC DSWM will be notified within 14 days of this finding. In accordance with Rule 0400-11-01-.04(7)(a)5, Scepter may within 90 days from the sample analysis date, demonstrate that a source other than the landfill caused the contamination or that the statistically significant increase resulted from error in sampling, analysis, and/or statistical evaluation, or from natural variation in groundwater quality. If such a demonstration is not documented and approved by the DSWM within 90 days from the sample analysis date, then Scepter will initiate an Assessment Monitoring Program as specified in Rule 0400-11-01-.04(7)(a)6.

4.0 GROUNDWATER SAMPLING AND ANALYSES

The following section briefly summarizes the primary components of the groundwater sampling and analysis plan to be utilized at the Scepter Disposal Facility.

4.1 HEALTH AND SAFETY PLAN

Prior to the field sampling activities, the site Health and Safety Plan (HASP) will be updated specifically for current operations at the Scepter landfill in Waverly, TN. The HASP is designed to assign responsibilities, establish personal protection standards and mandatory safety procedures, and provide for contingencies that may arise while groundwater monitoring activities are being conducted at the site. The HASP complies with, but does not replace, Federal Health and Safety Regulations, as set forth in 29 CFR 1910 and 1926, and applicable state regulations. This HASP is to be used by field personnel as a supplement to these rules, regulations, and guidance.

Any proposed changes to the plan will be reviewed with a health and safety professional prior to their implementation. If this is not feasible, the Site/Project Manager may modify the plan and record all changes in the field logbook. Under no circumstances will modifications to this plan conflict with federal, state, or other governmental health and safety regulations.

4.2 GROUNDWATER LEVEL GAUGING

Each sampling event will commence with a site-wide groundwater level gauging event to provide a “snapshot” of groundwater head distribution. The depth to water surface from the top of each reference point (e.g., top of well casing) will be measured in each well to the nearest 0.01 foot with an electronic water level indicator before pumping or bailing begins, and the total depth of the well will be measured using a weighted tape. The volume of water present in the well prior to sampling will be calculated and recorded along with other well measurements and observations on the Groundwater Data Field Worksheet. The water level indicator will be cleaned after each measurement by rinsing with distilled water and wiping dry as it is wound on the reel.

4.3 GROUNDWATER PURGING AND SAMPLING

Wells will be purged prior to sampling to ensure that representative groundwater is obtained from the uppermost aquifer. Either the low-flow minimal drawdown or volume-averaging purging methods will be performed. Low-flow sampling will be performed in general accordance with USEPA published protocols (USEPA 1996A and 1996B) or the American Society for Testing and Materials standard practice (ASTM, 2002). Volume-averaging purging will be performed by removing a minimum of three columns of water from a well using a variable-speed submersible pump or bailer.

For low-flow sampling, field parameters will be continuously monitored while purging using a calibrated, in-line, multi-parameter, flow-through cell. The field instruments used to collect water quality data will be appropriately calibrated each day in accordance with the manufacturer’s instructions. Sample collection will begin after stabilization of the field parameters. Parameter stability will be defined as three successive readings taken at 3- to 5-minute intervals that are within ±0.1 for pH, ±3% for conductivity, ±10% for turbidity (or less than 10 NTU), and ±3% °C for groundwater temperature. For the volume-averaging purging method, field parameters will be measured periodically during purging in an open sample container using a calibrated multi-parameter water quality meter, but purging will be considered complete upon the removal of three columns of water from the well.

Time, purge rate, and groundwater level will be periodically recorded throughout the purging operation. Well purge water will be handled in accordance with applicable investigation derived waste (IDW) protocols and regulations.

If a low yielding well is purged dry, the well will be allowed to recharge for up to 24 hours. If within this period, sufficient water is present in the well to obtain the necessary sample volume, a sample will be collected using a new disposable Teflon bailer or pump.

Samples will be collected directly from the pump discharge line (or disposable bailer) in new certified sample containers containing appropriate preservatives (where applicable). Clean nitrile (or equivalent) gloves will be worn when handling sample containers and the sampling equipment. When filling sample bottles, care will be taken to minimize sample aeration and overfilling. Sample bottles will be filled one at a time and capped before filling the next bottle. Samples will be placed on ice immediately after collection.

All sample containers will be labeled with permanent sample identifications (ID). This sample ID number will be unique for each sample collected and will be cross referenced on all field sheets and on the sample chain of custody (COC) form.

Any problem observed that might affect the quality of these procedures will be identified and recorded on the field data sheet, along with the action(s) taken to resolve it. Problems that might affect quality include clogged sampling tubes, highly turbid samples, defective material or equipment, inability to comply with quality procedures, or atmospheric/ambient conditions.

4.4 Sampling Equipment Decontamination Procedures

Sampling equipment will be decontaminated prior to use in the first well and following sampling of each subsequent well. Pumps will not be removed between purging and sampling operations. The pump and tubing (including support cable and electrical wires which are in contact with the well) will be decontaminated by one of the following procedures:

- Flush the equipment / pump with potable water.
- Flush with non-phosphate detergent solution. If the solution is recycled, the solution must be changed periodically.
- Flush with tap or distilled / deionized water to remove all of the detergent solution. If the water is recycled, the water must be replaced periodically.
- Flush with distilled / deionized water. The final water rinse need not be recycled.

4.5 QUALITY ASSURANCE AND QUALITY CONTROL

Four types of QA/QC samples will be utilized to meet the project. QA/QC samples will be collected only for samples undergoing chemical analysis.

- Trip Blanks

  A blank that is prepared in the laboratory, transported to the sampling site, and handled in the same manner as other samples, except that it remained unopened. If VOC's are being analyzed in the groundwater samples, the trip blank is returned to the laboratory for VOC analysis to ensure that contamination is not introduced to samples via transportation or handling procedures.

- Equipment Rinsate Blanks

  A blank prepared in the field using deionized/distilled water provided by the laboratory. The water is poured over/through sampling equipment that has been previously decontaminated. The blank water is then collected into sample bottles and analyzed for the parameters of interest. The purpose of this blank is to ensure that field conditions and/or the equipment are not introducing contaminants to the samples.

- Field Duplicates

  A duplicate sample taken in the field and sent to the laboratory for analysis. The results will provide some indication of the homogeneity of the sample medium and the precision of the field sampling and laboratory sample analysis. Accurate field notes will ensure that each duplicate can be matched to its corresponding investigatory sample.

- Matrix Spike/Matrix Spike Duplicates (MS/MSDs)

  A "MS" is a subsample of an investigatory sample to which the laboratory adds a spike containing analytes at known concentrations prior to extraction/analysis of the sample to assess the effect of sample matrix on the extraction and analysis methodology. The MSD is another subsample from the original investigatory sample (subsampling performed at the laboratory) which is similarly spiked. The sample selected for MS/MSD analysis will not be from the location from which the field duplicate is obtained and will be indicated on the chain-of-custody. Additional sample volumes will be collected to perform these analyses.

QC samples will be identified on the sample container label and on the chain-of-custody documentation. Field duplicates, MS/MSDs, rinsate blanks, and trip blanks (if applicable) will be collected at the frequency shown in the table below for each parameter class of interest.

QC Sample	Frequency
Trip Blanks	1/day/cooler containing VOC samples
Field/Equipment Rinsate Blanks	1/week/matrix or 1/20/matrix*
Matrix Spikes/Matrix Spike Duplicates	1/20/matrix

Field Duplicate	1/day/matrix or 1/20/matrix*

*Whichever is less

The contracted laboratory will maintain detailed records of analytical procedures for a minimum of five years, in order to support the validity of the analytical work. Each data report submitted to the Project Manager will verify that the approved analytical method was performed and that QA/QC checks were within the established protocol limits on the samples. The verification must be provided by the laboratory project manager, laboratory manager, or QA officer. If any QA problems are encountered during sample analysis, the laboratory Project Manager and/or QA Officer will report the problem(s) to the URS Project Manager in writing. This may be in the form of a technical narrative in the data package. The laboratory generates internal reports for evaluating and documenting the quality assurance program. These reports may include:

- The results of internal systems audits including any corrective actions taken;
- Performance evaluation scores and commentaries;
- Results of site visits and audits by regulatory agencies and clients;
- Performance on major contracts (including CLP as applicable);
- Problems encountered and corrective actions taken;
- Holding time violations; and
- Comments and recommendations.

4.6 Sample Preservation and Shipment

All samples will be adequately marked for identification (ID) from the time of collection and packaging through shipment and storage. Marking will be on a sample label attached to the sample container using a waterproof pen designed for that purpose. The information on the sample label will include, at a minimum, the project name, sample ID number, requested analysis, sample date and time, and initials of the individual(s) performing the sample collection.

The amount of sample retained is dependent on the analysis required. Each groundwater sample collected will be transferred from the sampler to new, clean, laboratory-provided containers appropriate for the anticipated analysis. The top of each container will be immediately closed tightly to seal it and to prevent loss due to volatilization.

Any samples, sample vials, or empty sample or storage bottles containing methanol as a preservative will be segregated from all other sampling containers, and shall not be shipped or stored in any coolers containing sample containers intended for VOC analysis. The minimum storage requirements will be as follows:

- Store and ship in an insulated cooler nominally at 4 °C;

- Samples will be protected from freezing by storing in a temperature regulated space where necessary; and
- All samples will be stored in a safe place under strict chain of custody to preclude any loss or disturbance.

4.7 Chain of Custody Control

Samples are physical evidence collected from a site or the environment. An essential part of any sampling and analytical scheme is ensuring the integrity of the sample from collection to data reporting. This includes the ability to trace the possession and handling of samples from the time of collection through analysis and final disposition. A sample is considered to be in custody under the following conditions:

- It is in the person's physical possession;
- It is in view of the person after he/she has taken possession;
- It is secured by that person so that no one can tamper with the sample; or
- It is secured by that person in an area, which is restricted to authorized personnel.

All environmental samples will be handled under strict COC procedures, beginning in the field. The designated Field Team Leader will be the field sample custodian and will be responsible for ensuring that the procedures are followed. Sample custody for field activities will include the use of COC forms, sample labels, custody seals, and field notebooks. Field notebooks will be used throughout the project to document all phases of field activities. Supplies and reagents used for field measurements will be recorded in the field notebooks or field data sheets.

When the COC is initiated at the laboratory, the laboratory personnel responsible for shipping sampling containers will have initiated and signed the COC form and sealed the shipping container with a COC seal. The field staff should acknowledge receipt and container integrity by signing the COC form, noting any discrepancies.

If custody of the samples (and sample bottles) is exchanged during field sampling, such transfer must be documented on the COC form. The departing field staff should sign indicating the custody has been relinquished, and the arriving field staff should sign indicating responsibility for the custody of the samples.

4.8 Laboratory Analyses

The unfiltered groundwater samples will be analyzed for the inorganic constituents listed in Section 3.1. Analytical methods will be approved EPA methods per EPA Publication SW-846. The particular SW-846 method used will have laboratory reporting limits of the lowest practical quantitation limits (PQL) (i.e., laboratory reporting limits) that can be reliably achieved within specified limits of precision and accuracy.

4.9 Water Quality Goals

Where possible, analytical data will be compared to the Environmental Protection Agency's (EPA) maximum contaminant levels (MCLs) for primary and secondary drinking water standards presented in the following table:

PMCL (mg/L)

Arsenic	0.010
Barium	2
Beryllium	0.004
Cadmium	0.005
Chromium	0.1
Mercury	0.002
Nitrate	10
Selenium	0.05

SMCL (mg/L)

Aluminum	0.05 - 0.2
Chloride	250
Copper	1.0
Total Dissolved Solids	500
Iron	0.3
Manganese	0.05
Silver	0.1
Sulfate	250
Zinc	5

PMCL - Primary Maximum Contaminant Level

SMCL - Secondary Maximum Contaminant Level

4.10 Record Keeping

Field logs will be maintained by Scepter or their representative. The field data sheets will be used to record pertinent data and observations for each sampling event. Copies of the field logs and data sheets will be maintained in the project file. Examples of representative field forms for sampling activities are included in Appendix A.

5.0 REFERENCES CITED AND BIBLIOGRAPHY

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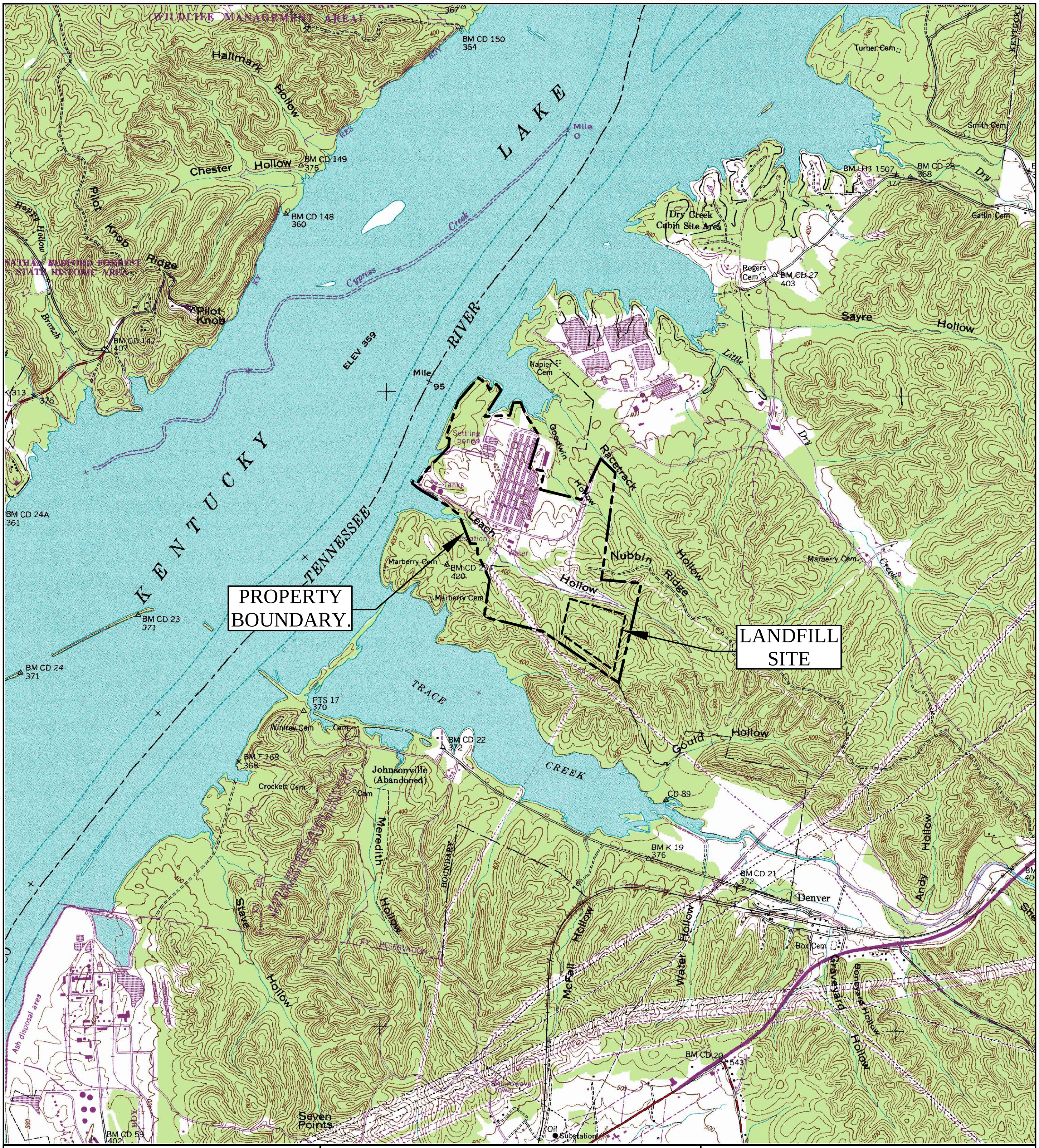
FIGURES



USGS QUAD MAP
JOHNSONVILLE, TN
SCALE 1:24000

N

SCEPTER, INC.
Waverly, TN

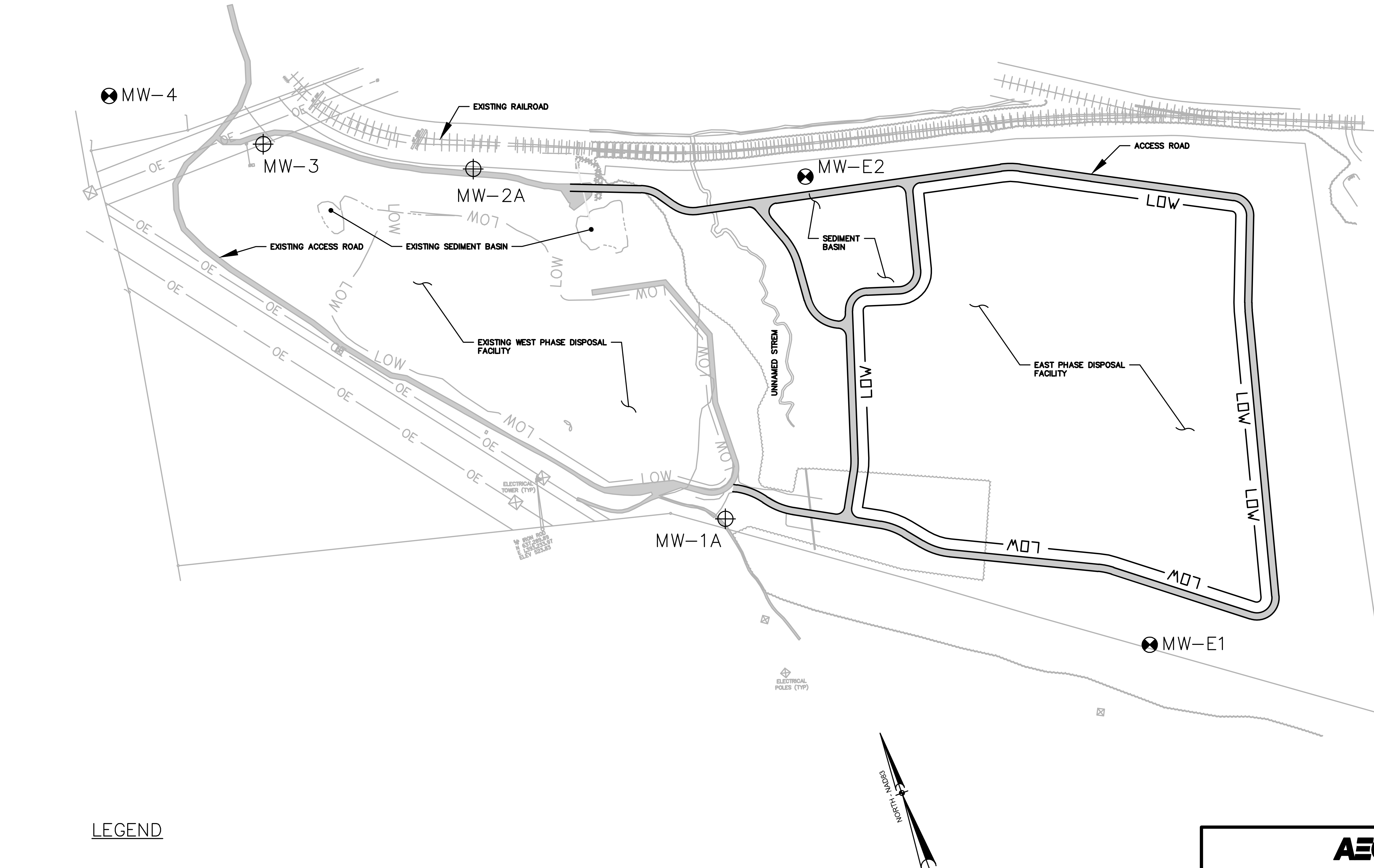
Date: 2/10/16	Project No. 60398526
Drawn By: MRM	Checked By: MSM

AECOM
1000 Corporate Centre Drive, Suite 250
Franklin, TN 37067

EAST PHASE SITE
HYDROGEOLOGIC
INVESTIGATION

Figure 1
SITE LOCATION MAP

S:2016\Scepter\028 Waverly\02 Landfill\Hydrogeo\Figures\1-1\Figure 1.dwg



LEGEND

MW-E1	2016 BOREHOLE/M.W.
	EXISTING M.W.
	BENCHMARK
— LOW —	LIMITS OF WASTE

0 300' 600'

GRAPHIC SCALE

AECOM

SCEPTER, INC.

WAVERLY, TENNESSEE

MONITORING WELL LOCATIONS

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	FIGURE
NP	MM	60398526	7/1/16	2

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APPENDIX

APPENDIX C.1:Field Worksheet

AECOM

Groundwater Monitoring Program
Gauging Log & Development / Sampling Log

Well:	**Project: Scepter Waverly Landfill**	**Purge Method:** Low-Flow Purge
Date:	**Project #:**	**Purge Equipment:** QED Bladder Pump/ **Calibration:** date/time ____________
Gauging Data from Top of Casing (Reference Point)		
Depth to Water: feet	**Total Well Depth: feet**	**Water Column Height: feet**
Gauging Time:	**1 Casing Volume: gallons**	**3 Casing Volumes: gallons**

Depth to Water	Time	Purge Rate	Vol. Purged (ml)	pH (Units)	Specific Conductivity (µS/cm)	Turbidity (NTU)	Temp. (^oC)	DO (mg/L)	ORP (mV)	Notes
		ml/minute								Start Purging

Sample ID.(time): ____________( ) Dup ID.: ____________( ) Equipment Blank ID.: ____________( )

Visual Inspection of well; lock, casing, concrete pad, well ID (✓) ______ Sampler: ____________

Comments: ____

____

PART D
CONSTRUCTION QUALITY ASSURANCE PLAN

PART II PERMIT APPLICATION
SCEPTER, INC. DISPOSAL FACILITY – EAST PHASE SITE
CONSTRUCTION QUALITY ASSURANCE PLAN

Prepared for
Scepter, Inc
Waverly, TN

July 1, 2016
April 5, 2018 Rev 2

Prepared by

AECOM

TABLE OF CONTENTS

AECOM

Tables

1.0 INTRODUCTION

This Construction Quality Assurance (CQA) Plan provides direction for the construction of the Scepter, Inc. Class II waste disposal landfill located in Waverly, Tennessee. The facility is owned and operated by Scepter, Inc. (Scepter).

This CQA Plan has been prepared in accordance with the requirements of Rule 0400-11-01-.04(9)(c)19 as issued by the Tennessee Department of Environment and Conservation's (TDEC) Division of Solid Waste Management (DSWM). The purpose of this Plan is to establish standards that, when followed by the CQA Inspection Personnel, will ensure that the proposed landfill is constructed in accordance with the Engineering plans. The elements of construction that require field monitoring and documentation under the Plan include compacted soil liner, geosynthetic clay liner, geomembrane liner, leachate collection system and final cover. In addition, field monitoring and documentation of subgrade inspection and buffer verification will also be required.

The purpose of this Construction Quality Assurance (CQA) Plan is to outline the observation and testing requirements needed to document and verify the following:

- How each new "as built" solid waste disposal facility unit(s) liner(s) and/or lateral expansion liner(s) and cover system(s) will be inspected and/or tested by a registered engineer as required at rule 0400-11-01-.04(1)(c) during construction or installation for uniformity, damage, and imperfections;
- How each constructed section of the liner system or final cover system will be certified by a registered engineer; and
- The engineered components meet the lines and grades shown in the construction drawings and conform to the specifications in the construction documents. The engineered components meet the lines and grades shown in the construction drawings and conform to the specifications in the construction documents.

2.0 QA/QC PROGRAM

Although they are related, there is a significant difference between Quality Assurance and Quality Control. These elements are often confused and interchanged because they are interdependent. Quality Assurance (QA) relies on the Quality Control (QC) feedback and both work to deliver good quality products and services. Although this plan is focused on construction quality assurance, it necessarily also contains items associated with quality control.

2.1 ROLES AND RESPONSIBILITIES

2.1.1 OWNER AND OPERATOR

The plant and its ancillary functions are owned and operated by Scepter, Inc. The Owner will be responsible for overall management of construction activities including contracting and administration.

2.1.2 CONTRACTOR

The Contractor for this project will be selected by the Owner. The Contractor is responsible for construction activities associated with this project including meeting all of the requirements for project quality as defined in the construction plans and specifications for his work as well as that of his Subcontractors.

2.1.3 CQA CONSULTANT

The CQA Consultant is responsible for making observations and performing field tests to provide written documentation that a facility is constructed in accordance with the applicable plans, and specifications and CQA Plan. The CQA Consultant may contract with third party testing firms to conduct on-site and laboratory testing, as necessary. The CQA Consultant is responsible for preparing the Construction Certification Report and record drawings for the project. The following provides a description of the typical CQA Consultant team, including each member's roles and responsibilities.

2.1.4 CERTIFYING ENGINEER

The CQA Certifying Engineer is responsible for certifying to the Owner and the permitting agency that the facility has been constructed in accordance with the plans, drawings, and the approved CQA Plan. The Certifying Engineer serves as the Professional Engineer for the project and properly certifies the as-built construction record document. Certifications will bear the seal of a Professional Engineer registered in the state in which the work is being performed. The Certifying Engineer may also function as the Project Engineer.

2.1.5 PROJECT ENGINEER

The CQA Project Engineer is responsible for providing engineering and technical support to the field CQA team throughout the construction process. The Project Engineer works closely with the CQA Inspector to assist with calculations and complete take-offs in support of as-built quantities for payment. The Project Engineer also reviews submittals and Requests For Information (RFI) from the Contractor, reviews and maintains QA/QC data, and coordinates all supplementary laboratory testing of geosynthetics and soils. The Project Engineer will provide the following on-site QA personnel as needed:

- CQA Inspector
- Third-party CQA testing firm
- Third-party surveying firm

2.1.6 CQA INSPECTOR

The CQA Inspector will observe and document construction activities for compliance with the contract documents. Specific duties of the CQA inspector include:

- Observe and document all construction related activities.
- Observe and document geosynthetic installation activities.
- Coordinate testing with CQA Subcontractor
- Monitor delivery, handling and on-site storage of construction materials.
- Evaluate conformance of all borrow source materials.
- Observe material placement and testing.
- Observe the installation and testing of all mechanical and electrical systems.
- Coordinate material sampling and shipping for laboratory testing.

2.1.7 SUBCONTRACTORS

The CQA Consultant will subcontract with a construction materials testing and inspection firm for field and laboratory testing as needed. The CQA Subcontractor will provide technicians for testing and observance including:

- Drilling and subsurface exploration
- Specialty geotechnical services
- Submersible pump installation

Laboratory testing may include:

- Soil testing
- Concrete testing
- Geosynthetic testing

2.2 PROJECT MEETINGS

To achieve a high degree of quality during installation, clear, open channels of communication are essential. The following meetings should be held when appropriate.

- Pre-Construction Meetings

Two Pre-Construction Meetings may be held, one prior to earthwork construction and one prior to geosynthetic placement.

- Daily Meetings

A daily meeting will be held, as necessary, between the CQA Consultant, the Geosynthetic Installer, and other involved parties. Those attending will discuss, plan, coordinate the work, and CQA activities to be completed that day.

- Progress Meetings

Progress meetings will be held routinely as determined to be necessary by the CQA Consultant and owner.

2.3 DOCUMENTATION

An effective CQA plan depends largely on recognition of construction activities that should be monitored, and on assigning monitoring responsibilities. This is most effectively accomplished and verified through quality assurance activities. The CQA Consultant will document that quality assurance requirements have been addressed and satisfied.

The CQA Consultant will prepare and provide to the Owner periodic signed reports which summarize construction activities and the results of observations and tests including descriptive remarks, data sheets, and logs to verify that all quality assurance monitoring activities have been carried out.

2.3.1 CONSTRUCTION CERTIFICATION REPORT

At the completion of the work, a signed Construction Certification Report will be submitted by the CQA Consultant prepared in accordance with the project requirements

The Construction Certification Report will be prepared and signed and sealed by a professional engineer skilled in the appropriate discipline(s) and registered in the state of Tennessee.

At a minimum, the Construction Certification Report will include:

1. A narrative section that identifies the engineered components that were constructed that includes the following:
    - A summary of the design and construction specifications and a comparison with the components that were constructed during the construction event
    - A summary of how construction was impacted by weather and equipment limitations and other difficulties encountered
2. All alterations and other changes that relate to the installation of any of the components to be certified and presented as follows:

- A listing of all applicable alteration requests/changes that were previously concurred with
- All alteration requests/changes and supporting documentation which are proposed for concurrence
- A list of any other changes made by the owner or operator which do not require regulatory concurrence but which affect construction or the record drawings

The alteration request will be equivalent or more protective than the applicable regulation or authorizing document.

3. Results of all tests in accordance with the project specifications.
4. Results of all surveys in accordance with the project specifications. Unless otherwise specified, the survey data will be reported in a table(s) displaying the northing and easting for each designated survey point established to be no more than one hundred feet apart based on the grid system coincident with the design drawings. Additional points will be established at grade breaks and other critical locations.
5. Record drawings of the constructed facility components showing the following:
   - The location of all survey control points.
   - Plan views with topographic representation of all engineered components depicted along with critical elevations such as pipe inverts, sump elevations, ditch flow lines, tops and toes of berms, locations of repairs, etc.
   - The location and as-built detail drawings of all components to be certified.
   - If the Certification Report is submitted for the composite final cover system, cross sections showing the top elevations of the existing waste, top elevation of the composite cap system, and the elevations of the surface water management system. The cross sections will be taken at the same locations and using the same scale as in the approved permit to install. Otherwise, the cross sections will be taken at an interval no greater than every three hundred feet of length and width.
   - If the Certification Report is submitted for establishment of facility survey marks, the following information summarizing the activities performed to construct and establish the facility survey marks:
     - An identification and description of the known control point(s) used to establish the horizontal and vertical coordinate(s) of the facility survey marks.
     - The horizontal and vertical coordinates of the known control point(s) and facility survey marks.

- A summary of surveying activities performed in determining the coordinates of the facility survey marks.
- A copy of the 7.5 minute series quadrangle sheet(s) used in establishing the survey marks with the known control point(s) and the location of the facility survey marks clearly identified.
- A detailed drawing(s) illustrating the design of the facility survey marks, as constructed.

6. Qualifications of testing personnel that provided construction oversight and conducted all the testing on the engineered components for which the Certification Report is submitted including a description of the experience, training, responsibilities in decision making, and other relevant qualifications.
7. A notarized statement that, to the best of the knowledge of the owner or operator, the Certification Report is true, accurate, and contains all information required by this rule and by the CQA plan.

2.4 Failed Test Procedures and Alterations

A "failed test" occurs when a test performed on an engineered component yields a result that does not meet the specifications outlined in the applicable construction drawings or specifications. Testing performed on an engineered component which does not meet the specifications is not considered a failed test if the engineered component is undergoing construction or installation at the time of testing and the testing is performed for the purpose of gauging the effectiveness or completeness of construction. An “alteration” or “field change” is a change in construction materials, specifications, or CQA procedures from the project requirements that is necessary to perform the work or meet project requirements.

2.4.1 Failed Test Prior To Certification Report Submittal

If, prior to submission of the Construction Certification Report for the engineered component, the CQA Consultant determines that there is a "failed test," the CQA Consultant will perform all the following:

- Retest or otherwise assess the engineered component or portion of the facility to determine if construction is in compliance with the construction plans and specifications or other project requirements and include the final results in the Certification Report.
- Implement measures to attain compliance with the construction plans and specifications or other project requirements. An area with a verified failure must be reconstructed. Reconstructed areas must be retested at a frequency acceptable to

the CQA Consultant and at a frequency and location(s) sufficient to demonstrate that compliance has been achieved.

2.4.2 Alteration Prior to Submittal of Certification Report

If, prior to submission of the Construction Certification Report the CQA Consultant and/or Certifying Engineer determines that an alteration or field change is necessary to the construction drawings or specifications, the CQA Consultant will do all of the following:

- Include the applicable testing results and an assessment and justification for the necessary change(s) in an appropriate section of the Certification Report where the change is clearly identified.
- Provide a demonstration in the Certification Report that the change(s) are at least equivalent to the project requirements, the construction plans and specifications, and are at least as protective to human health and the environment.
- Submit the Certification Report as required.

2.4.3 Detection of the Change after Submittal of the Certification Report

If, after submission of the Construction Certification Report the CQA Consultant and/or Certifying Engineer determines that the Certification Report is in error due to improper documentation of an alteration or field change of the construction drawings or specifications, the CQA Consultant will do all of the following:

- Notify the Owner as determined by the Certifying Engineer of the change within twenty-four hours after discovery, by phone and within seven days after discovery in writing.
- Within fourteen days of submitting the written notification required above, do either of the following:
  a. Implement the failed test procedures outlined above (2.6.1) and amend and resubmit the Construction Certification Report to explain the circumstances and how compliance was achieved.
  b. Submit the Alteration information outlined above (Section 2.6.2).

2.5 CQA Consultant Documentation

Appropriate construction documentation forms will be approved for use by the Certifying Engineer.

2.6 Surveying

Surveying of lines and grades will be conducted on an ongoing basis during construction of soil layers, geosynthetics placement, and other engineered components. Surveying will be performed to provide documentation for record plans, verifying quantities, and assist the

Contractor in complying with the required grades. Surveying conducted at the site must be part of the CQA program.

The purpose of the survey is to verify that actual thickness and grades of the construction components are in accordance with the plans and specifications. Surveying of lines and grades will be conducted during construction of the soil layers. Surveying will be performed to provide documentation for record plans, verify quantities of soils and geosynthetics, and assist the Contractor in complying with the required grades. Review of the surveys conducted at the site will be part of the CQA program. The permanent benchmarks at the facility will be used for survey control. Surveying will be performed under the supervision of a qualified, professional Land Surveyor licensed in Tennessee.

Based on the control points provided by the Owner, the Contractor is to provide all temporary and permanent benchmarks, monuments, and increments needed to control work. If during the work, control points set by the Owner are disturbed by the Contractor, the Contractor will replace the control points.

2.6.1 SURVEY CONTROL

The permanent benchmarks at the facility will be used for survey control. One or more temporary benchmarks will be established for the site at a location convenient for daily tie-in. Temporary benchmarks are to be as accurate as third order benchmarks. The vertical and horizontal controls for this benchmark will be established within normal land surveying standards.

All benchmarks established at the facility will comply with the following requirements:

- At least three permanent survey marks, with each located on separate sides of the facility, will be established prior to any construction and within easy access to the limits of solid waste.
- Survey marks will be referenced to the same horizontal (TN State Plane NAD83) and vertical datum (National America Vertical Datum NAVD88) used on the design plans.
- Survey marks will be at least as stable as a poured concrete monument 10-inches in diameter installed to a depth of 42-inches below the ground surface. Each constructed survey mark will include a corrosion resistant metallic disk which indicates horizontal and vertical coordinates of the survey mark and will contain a magnet or ferromagnetic rod to allow identification through magnetic detection methods.

Survey control standards for the survey marks will be in accordance with the following:

- For the first facility survey mark established from the known control point, minimum horizontal distance accuracy will be one foot horizontal to two thousand five hundred feet horizontal (1’ Horiz : 2500’ Horiz).

- For each facility survey mark established from the first facility survey mark, minimum horizontal accuracy will be one foot horizontal distance to five thousand feet horizontal (1' Horiz : 5000' Horiz).
- For the first facility survey mark established from the known control point and for each facility survey mark established from the first facility survey mark, minimum vertical accuracy will be one inch to five thousand feet horizontal (1" Vert : 5000' Horiz).

2.6.2 Precision and Accuracy

The survey instruments used for this work will be precise and accurate to meet the needs of the project. Survey instruments will be capable of reading to a precision of 0.01 of a foot (3.1 mm) and with a setting accuracy of 10 seconds.

A vertical tolerance of ±0.01 feet will apply to each of the following components as they are constructed:

- Surface of excavation or top of structural fill
- Top of geologic buffer layer/bottom of compacted soil liner
- Top of compacted soil liner
- Top of protective cover layer
- Top of cap system subgrade/bottom of cap system
- Top of soil cover layer
- Invert of pipes

Note: These tolerances are meant to assure that the required layer thickness and design intent can be met upon final certification. A Professional Surveyor registered in Tennessee will certify results of the survey. Results will be included in the Certification Report provided to the Owner.

2.6.3 Frequency and Spacing

Surveying will be performed as soon as possible after completion of a given installation to facilitate progress and avoid delaying the next installation. In addition, spot checks during construction will be necessary to assist the Contractor in complying with the required grades.

The as-built thickness of various components of the facility (protective covers and compacted soil liners) will be determined by non-destructive methods, i.e., comparison of the survey data for the underlying materials with that of the component of interest. As-built survey data will be obtained at locations having a typical on center spacing of 100-feet maximum, at all toe, midpoint, and top of slope locations as well as grade breaks. Locations will be, to the maximum extent possible, at the same coordinates as the survey data for the underlying materials.

2.6.4 As-Built Milestone Surveys for Record Documentation

The Contractor's Professional Land Surveyor shall perform surveys to record as-built lines and grades of the work for Record Documentation. The following surfaces shall be surveyed to determine the lines and grades achieved during construction:

Final Cover System

Milestone 1. Cap subgrade

Milestone 2. Geomembrane Panel Placement including anchor trench location, limits of geosynthetic components, and destruct sample locations for the geomembrane.

Milestone 3. Top of two foot protective/vegetative soil layer

Milestone 4. Gas collection system vents, alignment and inverts of gas Collection piping

Liner System

Milestone 1. Liner System subgrade

Milestone 2. Top of two foot compacted soil liner

Milestone 3. Geomembrane Panel Placement including anchor trench location, limits of geosynthetic components, and destruct sample locations for the geomembrane

Milestone 4. Leachate Collection System including alignment and inverts of leachate Collection piping

Milestone 5. Top of Protective Cover

Milestone 6. Final Site Conditions including final topography of areas outside of the cell limits, the alignment and elevations of underground utility relocation, alignment and inverts of forcemain piping, valve box locations, electrical lines and components, and the alignment and inverts of storm water structures

The Milestone Surveys will serve as CQA Hold Points for the project, to verify that the lines, grades, and minimum thicknesses have been met for each component of landfill construction, and also to confirm that Quality Control and Quality Assurance documentation has been completed in accordance with this CQA Plan prior to constructing the engineered component associated with next Milestone survey.

When required, the extent of the following components will also be surveyed to determine the lines and grades achieved during construction:

- Leachate collection and conveyance piping including alignment, inverts, valve locations, and termination points.
- Surface water structure details including profiles, cross sections*, and inverts for ditches, culverts, catch basins, swales, benches, ditches, and sedimentation basins
- Alignment and inverts of piping (both inside and outside the facility)
- As-built line and grade of all other piping structures
- Roads including profiles and cross sections with a minimum of one cross section for every 300 linear feet of waste containment area will be surveyed.

2.6.5 Surveying Personnel

Surveying for construction certification and record documentation purposes will be performed under the supervision of a qualified, licensed Professional Land Surveyor registered in the state of Tennessee. The survey crew will consist of a Senior Surveyor and as many Surveying Assistants as required to satisfactorily undertake the work. Surveying personnel will be experienced in the provision of these services, including detailed, accurate documentation.

2.6.6 Certification

Survey results will be certified by a licensed Professional Land Surveyor licensed in the state where the work is performed and submitted to the CQA Consultant for review.

2.6.7 Surveys by Owner or Engineer

The Owner or Project Engineer may request additional surveys to monitor, verify, or document the work.

2.7 Submittals

2.7.1 Submittal Procedure

Submittals include shop drawings, material data, and samples. Product data submittals, samples, and shop drawings are required to verify that the correct products will be installed on the project. The shop drawing submittal is a drawing or set of drawings produced by the Contractor, Supplier, Installer, Manufacturer, Subcontractor, or Fabricator typically for pre-fabricated components or construction procedures. The product data submittal usually consists of the manufacturer's product information. The sample submittal is a physical portion of a specified product, often required when several products are acceptable, to confirm the quality and aesthetic level of the material. The size or unit of sample material usually is specified.

Four copies of all submittals will be initially submitted to the Project Engineer by the Contractor for review. After review, one copy of the submittal will be returned to the Contractor. The Contractor will include a letter of transmittal along with each submittal and include the following information at a minimum.

- Owner's Name
- Project Name
- Contract No.
- Transmittal No.
- Specification Section or Drawing Reference

Shop drawings will be submitted well in advance of the need for the material or equipment for construction by the Contractor and with ample allowance for the time required for engineer review and to accept delivery of material or equipment afterward in accordance with the project schedule.

2.7.2 SUBMITTAL REVIEW

After the Project Engineer completes his review, submittals will be returned to the Contractor indicating whether or not the materials meet the project requirements along with further instructions.

3.0 PREQUALIFICATION TESTING

3.1 SUMMARY

The following section discusses the specific QA/QC requirements for prequalification conformance testing of the engineered components for the landfill facility. Any reference to a material or product conformance standard assumes use of the most recent published version of that standard.

3.2 PREQUALIFICATION CONFORMANCE TESTING

Prequalification testing is necessary to establish that the materials used for the engineered components conform to the minimum specifications contained in this document for each component. Some prequalification testing is generally necessary for any engineered component comprised in whole or in part of aggregate, recompacted clayey soil, or geosynthetics. Conformance testing is performed on representative materials obtained from the location of origin and results submitted prior to arrival on site unless otherwise directed by the Project Engineer.

For this CQA Plan, prequalification conformance testing is divided between material characteristics testing and shear strength testing. Shear strength related testing requirements are contained in Table 2. Material characteristics testing is contained in Tables 3 through 11 as applicable for each engineered component. Soil and aggregate laboratory and field testing methods are located in Table 1A and 1B. Minimum testing requirements and frequencies for soils and aggregates are located in Table 3, and for geosynthetic clay liners, Table 4. Tables 5 through 11 contain testing requirements for geosynthetic components.

3.3 ENGINEERED COMPONENTS REQUIRING PREQUALIFICATION CONFORMANCE TESTING

Engineered components that require some measure of prequalification are noted in the individual specifications under submittals. These materials are summarized as follows:

- Geologic Buffer
- Structural Fill
- Compacted Soil Liner
- Final Cover Protective Soil and Vegetation
- Baseliner Protective Cover Layer
- Geosynthetic Clay Liner
- Geomembrane
- Geocomoposite
- Geotextile
- HDPE Pipe
- Surface Water Structures

3.4 SUBMITTALS

Submit the results of all required prequalification testing including interface friction testing, to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to use of the material at the job site unless otherwise directed by the CQA Consultant.

4.0 GEOLOGIC BUFFER

4.1 GENERAL

4.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and construction of this material. Geologic buffer may be constructed in part or in whole similarly to structural fill, but is designed and constructed to create a physical buffer between the disposal facility liner system and groundwater. It is further distinguished from structural fill in that it has a hydraulic conductivity requirement.

4.1.2 DEFINITIONS

Geologic buffer refers to a layer of fill to establish a physical buffer between groundwater and the disposal facility liner system. This layer is of relatively low permeability, but is not designed to act as a leachate containment layer.

4.1.3 DESCRIPTION AND DESIGN REQUIREMENTS

The geologic buffer layer serves as a physical buffer between the disposal facility liner system and groundwater. In addition, the geologic buffer layer serves as a base layer for the installation of the overlying engineered components (e.g. compacted soil liner or geosynthetic clay liner).

4.1.4 SUBMITTALS

Submit the results of all required prequalification testing as required in Section 3, to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to use of the material at the job site.

Please note the specified grain size requirements and USCS classifications may be changed if the material is shown through testing to meet the minimum permeability requirements for the geologic buffer, and if accepted by the Project Engineer.

4.2 PRODUCTS

4.2.1 MATERIAL REQUIREMENTS

Obtain soil materials for construction of the prepared subgrade layer from on-site and/or off-site borrow sources or use in situ foundation materials as contained in the construction plans and/or as directed by the Project Engineer. Remove excess or unsatisfactory material to designated on-site stockpiles as directed by the Project Engineer or CQA Inspector.

Utilize material for the geologic buffer layer or in situ foundation with the following characteristics:

- Consists of well-graded natural earth material that is not excessively dry or saturated
- Free of cobbles, stones, rock, gravel or boulders greater than 4-inches in diameter

- Free of organic materials, debris, waste, frozen materials, vegetation, roots, and any other deleterious materials, and any materials that could damage or puncture overlying materials
- Meets one of the following USCS soil classifications: SC, SM, GC, CL, CH, or CL-ML
- Maximum hydraulic conductivity of 1 x 10^{-5} cm/sec under moisture and density conditions consistent with the conditions in which it will be placed as determined by laboratory testing in accordance with ASTM D5084.

Any additional requirements are contained in Table 3.

4.2.2 Source Quality Control

Perform additional field or laboratory tests as often as is necessary to document the material continues to meet the prequalification specifications contained in the Material Requirements section above.

4.3 Execution

4.3.1 Preparation

Over-excavate and remove all unsuitable soil located below the prepared subgrade layer or in situ foundation elevations until a competent, stable surface at a lower elevation is reached. Replace the material as necessary with suitable structural fill material as directed by the Project Engineer or CQA Inspector.

4.3.2 Construction and Installation

Construct the geologic buffer layer to the lines and grades shown on the construction drawings.

Prepare and process the material as necessary to achieve the required hydraulic conductivity as applicable as determined by ASTM D5084 and compaction requirements as determined by ASTM D698 and/or ASTM D1557 in accordance with the requirements contained in Table 3. Please note that the compaction and moisture requirements may change to achieve the required density and permeability due to changes in soil and field conditions.

Place the material in loose lifts compacted by a minimum of two one-way passes (up and back over same area) of a soil compactor and as otherwise required as noted in Table 3. Overlap the passes so the entire area where material is placed receives a minimum of two, one-way passes of the compaction equipment.

Prior to placement of the geologic buffer, remove any unsuitable materials encountered in the subgrade (e.g. organics, soft/loose soil, protruding cobbles and boulders, etc.) and fill the resulting voids with structural fill or geologic buffer material appropriately compacted.

Key-in soil material into existing and/or constructed slopes with 2 horizontal to 1 vertical minimum benches.

Prepare the final surface to be relatively smooth such that the surface is suitable for the overlying engineered component(s).

4.3.3 FIELD QUALITY CONTROL

Prior to placement of the overlying compacted soil liners, perform a proofroll over a representative area of the geologic buffer layer to verify the adequacy of the subgrade soils to support the design loads. Coordinate this activity such that it can be observed by the CQA Inspector or Project Engineer.

Perform the proofroll with a pneumatic tired vehicle weighing a minimum of 25 tons and in accordance with the specifications in Table 3. Proofrolling may require multiple overlapping passes, as directed by the CQA Inspector or Project Engineer.

If excessive pumping or rutting occurs during proof-rolling as determined by the CQA Consultant, rework the area affected by the failed proofroll, or undercut and remove the unacceptable material and reconstruct the affected areas until it passes the proofroll.

The CQA Consultant will document that the geologic buffer construction was completed in conformance with the construction documents prior to placement of the overlying engineered components.

4.3.4 FIELD QUALITY ASSURANCE

Quality assurance of the placement of geologic buffer will be by in-place density testing or proofroll in accordance with Table 3 and be performed by the CQA consultant in general accordance with ASTM D698 or D1557.

Acceptance of the proofroll and lift thickness will be visually observed by the CQA consultant. Total thickness of geologic buffer will be verified by as-built survey on a 100-ft maximum grid.

5.0 COMPACTED SOIL LINER (CSL)

5.1 GENERAL

5.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and construction of this material. Compacted clayey soil will be used as the soil component of a composite liner system used for the landfill liner system.

5.1.2 DEFINITIONS

Compacted soil liner (CSL) material shall be placed upon the approved, prepared subgrade surface as part of the composite liner system for the proposed landfill.

5.1.3 DESCRIPTION AND DESIGN REQUIREMENTS

The construction of the CSL shall meet the buffer soil fill requirements for material properties, preconstruction testing, construction and field testing. The upper 4" of the CSL soil liner is required to be free of rock greater than 2" in order to protect the overlying GCL. The CSL serves as a base layer for the installation of the overlying engineered components (e.g. geosynthetic clay liner or geomembrane).

5.1.4 SUBMITTALS

Submit the results of all required prequalification testing as required in Section 3, including interface friction testing, to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to use of the material at the job site.

5.2 PRODUCTS

5.2.1 MATERIAL REQUIREMENTS

Obtain soil materials for construction of the CSL from on-site and/or off-site borrow sources or use in situ foundation materials as contained in the construction plans and/or as directed by the Project Engineer. Remove excess or unsatisfactory material to designated on-site stockpiles as directed by the Project Engineer or CQA Inspector.

Utilize material for the CSL layer with the following characteristics:

- Consists of well-graded natural earth material that is not excessively dry or saturated
- Free of cobbles, stones, rock, gravel or boulders greater than 4-inches in diameter except the lift that is in direct contact with geosynthetics will have a maximum particle size of less than 2-inches
- Free of organic materials, debris, waste, frozen materials, vegetation, roots, and any other deleterious materials, and any materials that could damage or puncture overlying materials

- Meets one of the following USCS soil classifications: SC, SM, GC, CL, CH, or CL-ML
- Meets a required maximum permeability of 1 x 10-6 cm/sec when placed in combination with an overlying geosynthetic clay liner.

Any additional requirements are contained in Table 3.

5.2.2 Source Quality Control

Perform additional field or laboratory tests as often as is necessary to document the material continues to meet the prequalification specifications contained in the Material Requirements section above.

5.3 Execution

5.3.1 Preparation

Over-excavate and remove all unsuitable soil located below the prepared subgrade layer or in situ foundation elevations until a competent, stable surface at a lower elevation is reached. Replace the material as necessary with suitable structural fill material as directed by the Project Engineer or CQA Inspector.

5.3.2 CSL Construction Specifications

Construct the compacted soil liner using the following minimum specifications unless altered by the results of a surrogate test pad or as directed by the Project Engineer:

- Use compaction equipment manufactured for the purpose of compacting cohesive soils.
- Maximum loose lift thickness is 8 inches
- Maximum clod size of three inches or half the lift thickness, whichever is less
- Minimum soil moisture is equal to or greater than optimum moisture as determined by standard or modified Proctor relationships (ASTM D698 or ASTM D1557 as applicable)
- Minimum soil dry density is equal to or greater than 95% of the maximum standard proctor density or 90% of the maximum modified proctor density (ASTM D698 or ASTM D1557 as applicable)

5.3.3 CSL Placement

Place the material in loose lifts compacted by a minimum of four passes (up and back over same area, move over half a drum, then up and back over the same area again) of a soil compactor and as otherwise required as noted in Table 3. Overlap the passes so the entire area where material is placed receives the minimum number of, one-way passes of the compaction equipment. The total number of passes required should be as determined by the test pad to achieve the density required.

Scarify compacted each lift prior to placement of the next lift to promote lift bonding and prohibit equipment or truck trafficking on the soil surface between scarifying and placement of the following lift.

Keep the work area small enough to maintain moist soil conditions, facilitate bonding, and minimize desiccation and crusting of the lift surface. If desiccation and crusting of the lift surface occurs before placement of the next lift, scarify the area, adjust the moisture as necessary, compact, and retest the area before placement of a subsequent lift.

Ensure each lift is thoroughly compacted and satisfies the moisture and density requirements through field QA testing before a subsequent lift is placed.

Transition from full-depth CSL to the beginning of an adjacent new section by benching in each new lift as shown in the plan drawings or as directed by the CQA Inspector.

5.3.4 CSL FINAL SURFACE

Finish the surface of the CSL to prevent any abrupt changes in grade that may result in damage to the geosynthetics and ensure the surface is smooth and free of sharp objects, roots, sticks and other deleterious materials.

Remove any sharp stones or other hard objects in the top inch of the finished surface to be covered with a geomembrane.

5.3.5 FIELD QUALITY ASSURANCE

Quality assurance of the placement of CSL will be by in-place density testing in accordance with Table 3.. The locations of the individual tests must be adequately spaced to represent the constructed area. Any penetrations will be repaired using bentonite. Lift thickness and source verification will be visually observed and confirmed by the CQA Consultant. The CQA Inspector will observe that the finished CSL will have adequate strength to satisfy bearing capacity and slope stability strength requirements.

5.3.6 PROTECTION

At the end of each construction day's activities, seal completed lifts or sections of CSL by rolling with a rubber-tired or smooth drum roller with additional moisture added as necessary. Ensure sealed lifts remain moist and prevent desiccation by periodically applying water to the surface as necessary. Sealed areas must be properly scarified prior to placing the next lift of material. Protect the finished CSL from damage due to desiccation, freeze/thaw cycles, wet/dry cycles, and the intrusion of objects during construction and operation. Completed segments of the CSL may be protected from excessive moisture or desiccation by placing a protective cover such as polyethylene or other materials approved by the Project Engineer.

6.0 STRUCTURAL FILL

6.1 GENERAL

6.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and construction of this material. Structural fill will be used to establish base elevations in areas where undercuts are made in the subgrade, to raise site grades, to establish the bottom of the compacted soil liner or geologic buffer elevations, and to construct buttresses, berms and roads.

6.1.2 DESCRIPTION/DESIGN REQUIREMENTS

Structural fill typically serves as support and as a foundation for other engineered components or as replacement of unsuitable materials in discrete areas.

6.1.3 SUBMITTALS

Submit the results of all required prequalification testing as required in Section 3 to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to use of the material at the job site.

6.2 PRODUCTS

6.2.1 MATERIALS

Obtain soil or other acceptable material for construction of the structural fill from on-site and/or off-site borrow sources as contained in the construction plans and/or as directed by the Project Engineer. Remove excess or unsatisfactory material to designated on-site stockpiles as directed by the Project Engineer or CQA Inspector. Soil material removed from excavations may be reused as fill provided it meets the prequalification requirements listed herein.

Utilize material for structural fill with the following characteristics:

- Consists of well-graded natural earth material that is not excessively dry or saturated
- Free of cobbles, stones, rock, gravel or boulders greater than 4-inches in diameter except for the lift that is in direct contact with geosynthetics will have a maximum particle size of less than 2-inches
- Free of organic materials, debris, waste, frozen materials, vegetation, roots, and any other deleterious materials and any materials that could damage or puncture overlying materials
- Meets one of the following USCS soil classifications: GP, GM, GC, SP, SM, SC, SW, CL-ML, or CL

Additional requirements are contained in Table 3.

6.2.2 SOURCE QUALITY CONTROL

Perform additional field or laboratory tests as often as is necessary to document the material continues to meet the prequalification specifications contained in the Material Requirements section above.

6.3 EXECUTION

6.3.1 PREPARATION

Over-excavate and remove all unsuitable soil located below the structural fill layer or in situ foundation elevations until a competent, stable surface at a lower elevation is reached.

6.3.2 CONSTRUCTION AND INSTALLATION

Place structural fill to the lines and grades shown on the construction drawings.

Prepare and process the material as necessary to achieve the required minimum compaction requirements as determined by ASTM D698 and/or ASTM D1557 in accordance with the requirements contained in Table 3. Please note that the compaction and moisture requirements may change to achieve the required density due to changes in soil and field conditions.

Place the material in loose lifts compacted by a minimum of four passes (up and back over same area, move over half a drum, then up and back over the same area again) of a soil compactor and as otherwise required as noted in Table 3. Overlap the passes so the entire area where material is placed receives the minimum number of, one-way passes of the compaction equipment.

Remove any unsuitable materials encountered (e.g. organics, soft/loose soil, protruding cobbles and boulders, etc.) and fill the resulting voids with additional structural fill appropriately compacted.

Key in the structural fill into existing and/or constructed slopes with 2 horizontal to 1 vertical minimum benches.

Prepare the final surface to be relatively smooth such that the surface is suitable for the overlying engineered component(s).

6.3.3 FIELD QUALITY ASSURANCE

Quality assurance of the placement of structural fill will be by in-place density testing or proofroll in accordance with Table 3. The locations of the individual tests must be adequately spaced to represent the constructed area. Lift thickness and source verification will be visually observed and confirmed by the CQA Consultant.

7.0 FINAL COVER PROTECTIVE SOIL AND VEGETATION

7.1 GENERAL

7.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and construction of this material. Final cover protective soil will be used to establish final elevations within the limits of fill, serve as an isolation barrier, and provide erosion protection for the underlying material.

7.1.2 DESCRIPTION AND DESIGN REQUIREMENTS

A protective cover layer component of the final cover system will be placed over the geocomposite drainage layer. This layer serves to protect both the geocomposite drainage layer as well as the underlying geomembrane. The layer also provides adequate soil for the establishment of vegetation on the final cover. The upper 6-inches of the protective layer is referred to as the vegetative cover soil and the lower 18-inches is called the protective cover soil. The vegetative cover soil layer will be planted with vegetation to prevent erosion of the final cover system.

The barrier cover soil will be 6-inches thick with a maximum permeability of 1×10^{-6} cm/sec. This soil layer will be in contact with the overlying geosynthetics.

The intermediate cover soil is comprised of a 12-inch thick soil barrier and includes the 6-inch daily cover soil layer plus an additional 6-inch soil layer. This soil will be obtained from on-site borrow. Any rock material within this layer will be limited to 4” diameter in size.

7.1.3 SUBMITTALS

Submit the results of the required prequalification testing to the CQA representative as summarized in Table 3 for approval prior to excavation and/or stockpiling of the material for use.

7.2 PRODUCTS

7.2.1 MATERIALS

Obtain soil for placement of the final cover protective soil from on-site and/or off-site borrow sources as contained in the construction plans and/or as directed by the Project Engineer. Remove excess or unsatisfactory material to designated on-site stockpiles as directed by the Project Engineer or CQA Inspector. Soil material removed from excavations may be reused as protective soil provided it meets the prequalification requirements listed herein.

Utilize material for the final cover protective soil with the following characteristics:

- Consists of well-graded natural earth materials that are not excessively dry or saturated unless otherwise specified by the construction specifications

- Has sufficient fertility or can be amended to support vegetation in the top 12-inches of material
- Has a minimum thickness of 24-inches of material

Material used for the protective cover layer must be compatible with the geocomposite drainage layer.

Additional requirements are contained in Table 3.

Vegetation will be established on the final cover in accordance with the seeding, mulching, and fertilization specifications provided in Section 2.14 of the Operations Manual.

7.2.2 Source Quality Control

Perform additional field or laboratory tests as often as is necessary to document the material continues to meet the prequalification specifications contained in the Material Requirements section above.

7.3 Execution

7.3.1 Construction and Installation

Place the final cover protective layer to the lines and grades shown on the construction drawings. Place the protective cover layer only after the geosynthetics layers have been accepted in writing by the Project Engineer. Deposit and spread the protective cover soil in uniform lifts (12 inch maximum) using low ground pressure equipment. Complete the final cover soil protective soil layer such that it is well draining and exhibits a smooth uniform surface free from ruts, depressions, and debris.

Seeding of the final cover may begin after the area to be covered has been properly prepared and fertilized. Uniformly distribute seed to meet the application rate provided in the project specifications. Perform seeding only during periods of acceptable weather conditions.

Protect seeded areas with temporary erosion control matting (ECM) as shown in the construction drawings, or as necessary to prevent loss of seed and fertilizer. Complete all ECM field installation in accordance with the manufacturer’s recommended installation procedures and the construction specifications. Ensure that matting overlaps are shingled in the direction of flow.

7.3.2 Quality Assurance

Protective cover soil lift thickness and source verification will be visually observed and confirmed by the CQA Consultant. Total thickness will be verified by as-built survey on a 100-ft maximum grid.

The initial lift of material placed over the geosynthetics will be monitored by the CQA Consultant to assess the potential for damage to the underlying engineered components.

The CQA Consultant will verify the seed, application method, and application rates meet the construction specifications, and that seed has been uniformly distributed over the final cover.

8.0 BASE LINER SYSTEM PROTECTIVE COVER LAYER

8.1 GENERAL

8.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and construction of this material. A protective cover layer will be placed above the geocomposite drainage layer within the liner system.

8.1.2 DESCRIPTION AND DESIGN REQUIREMENTS

The protective cover layer will be a total of 2 feet thick and will consist of a one foot thick layer of sand or other approved aggregate and a 1 foot layer of soil. The layer will provide protective cover to the underlying geosynthetics, and will separate the geosynthetics from the above waste material. The protective cover is designed to remain permeable to allow leachate to reach the leachate collection system below. To ensure permeability of the soil component, full depth sand filled trenches will be constructed into the top 1 foot thick protective soil layer.

8.1.3 SUBMITTALS

Submit the results of the required prequalification testing to the CQA representative as summarized in Table 3 prior to obtaining material for use.

8.2 PRODUCTS

8.2.1 MATERIALS

Obtain sand for placement of the protective cover from on-site and/or off-site borrow sources as contained in the construction plans and/or as directed by the Project Engineer. Remove excess or unsatisfactory material to designated on-site stockpiles as directed by the Project Engineer or CQA Inspector.

Material used for the protective cover layer must be compatible with the geocomposite drainage layer. The material must be free of organic materials, debris, waste, frozen materials, vegetation, roots, and any other deleterious materials and any materials that could damage or puncture underlying materials.

Additional requirements are contained in Table 3.

8.2.2 SOURCE QUALITY CONTROL

Perform additional field or laboratory tests as often as is necessary to document the material continues to meet the prequalification specifications contained in the Material Requirements section above. Grain Size Distribution testing of the 1 foot sand material must be completed to document that the material is acceptable for use.

8.3 EXECUTION

8.3.1 CONSTRUCTION AND INSTALLATION

Place the protective cover layer to the lines and grades shown on the construction drawings.

Place the protective cover layer only after the geosynthetics layers have been accepted in writing by the Project Engineer.

Deposit and spread the protective cover in minimum 12-inch lifts using low ground pressure equipment.

8.3.2 QUALITY ASSURANCE

Lift thickness and source verification will be visually observed and confirmed by the CQA Consultant. Total thickness will be verified by as-built survey on a 100-ft maximum grid.

The initial lift of material placed over the geosynthetics will be monitored by the CQA consultant to assess the potential for damage to the underlying engineered components.

9.0 GEOSYNTHETIC CLAY LINER

9.1 GENERAL

9.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and installation of this material. Geosynthetic Clay Liner (GCL) will serve as a low permeability barrier from overlying liquids. It must be placed on a stable subgrade and may be used in conjunction with other geosynthetics to act as a composite liner or barrier system. GCL typically has extensive QA/QC requirements for the manufacturer, installer, and the CQA inspector.

9.1.2 DESCRIPTION AND DESIGN REQUIREMENTS

The GCL is designed to function as a liquid barrier for leachate and is used in conjunction with the geomembrane to form a composite liner system. GCLs are composed of a sodium bentonite material typically sandwiched between two needle punched geotextile fabrics and have a very low hydraulic conductivity.

9.1.3 SUBMITTALS

Submit the results of the manufacturer's quality control testing in accordance with the test requirements outlined in Table 4 to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the liner material to the job site.

Submit the results of all required prequalification testing as required in Section 3, including interface friction testing, to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the GCL material to the job site.

Upon delivery at the site, submit all roll identification and quality control certificates issued by the manufacturer to the on-site CQA consultant representative. Rolls without proper identification will be rejected.

9.1.4 QUALITY ASSURANCE

Prior to delivery of the rolls of GCL, the CQA Consultant will verify that representative samples are removed and forwarded to a qualified testing laboratory for testing to verify conformance with the test methods and values presented in Table 4.

9.1.5 DELIVERY STORAGE AND HANDLING

Handle the geosynthetic materials with due care and utilize handling equipment on-site that poses minimal risk of damage to the material.

9.2 PRODUCTS

9.2.1 MATERIALS

Utilize GCL for the disposal facility liner system that meets the requirements in Table 4.

9.2.2 SOURCE QUALITY CONTROL

Quality Control testing for the GCL will be performed by the manufacturer on the representative samples of the proposed material to demonstrate and verify the materials meet minimum requirements outlined in Table 4.

9.3 EXECUTION

9.3.1 CONSTRUCTION AND INSTALLATION

A daily Prepared Subgrade Acceptance form is to be signed and submitted by the installer to the Project Engineer.

Install the GCL above a stable subgrade after receipt of approval of the subgrade from the CQA Consultant. The GCL will be seamed per the manufacturer's directions.

Complete all geosynthetics field installation in accordance with the manufacturer's recommended installation procedures and the construction specifications as outlined in this Section and to the lines and grades shown on the construction drawings.

Record all construction details for all deployed GCL on individual forms acceptable to the Project Engineer.

Complete the installation such that the GCL is smooth, without wrinkles, tears, or holes, and covers the total surface of the disposal facility liner. Do not leave tools, debris, or surplus materials on the surface.

Cover all emplaced GCL panels with the overlying geomembrane within 24 hours and before any rainfall events or heavy dew.

9.3.2 QUALITY ASSURANCE

In addition to reviews of QC documentation, the Project Engineer or CQA Inspector will visually inspect installed GCL for damage and conformance with the Specifications.

The CQA Inspector and Project Engineer will confirm the roll identification corresponds to quality control certificates issued by the manufacturer.

The CQA Inspector will document observations during installation including damage, seaming methods (including overlap), repairs, laboratory test results, and conformance to specifications. Other items that will be noted include:

- The method and equipment used to unroll the panels does not cause damage to the GCL and does not damage the supporting soil
- The method used to place the panels for minimization of wrinkles

All field testing will be performed in the manner and at the frequency identified in Table 4.

10.0 GEOMEMBRANE

10.1 GENERAL

10.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and installation of this material. Geomembrane will serve as a low permeability barrier from overlying liquids. It must be placed on a stable subgrade and may be used in conjunction with other geosynthetics to act as a composite liner or barrier system. Geomembrane typically has extensive QA/QC requirements for the manufacturer, installer, and the CQA inspector.

10.1.2 REFERENCES

GRI-GM13 - Geosynthetic Research Institute: "Test Methods, Test Properties and Testing Frequency for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes"

GRI-GM14 - Geosynthetic Research Institute: "Selecting Variable Intervals for Taking Geomembrane Destructive Seam Samples Using the Method of Attributes"

GRI-GM17 - Geosynthetic Research Institute: “Test Properties and Testing Frequency for Linear Low Density Polyethylene (LLDPE) Smooth and Textured Geomembranes”

GRI-GM19 - Geosynthetic Research Institute: “Seam Strength and Related Properties of Thermally Bonded Polyolefin Geomembranes”

10.1.3 DESCRIPTION AND DESIGN REQUIREMENTS

The geomembrane (a.k.a. Flexible Membrane Liner or FML) is designed to function as a liquid barrier for infiltrating storm water to minimize the amount of contact water generated from surface water. FML is a stable material and retains its strength and hydraulic conductivity properties even after being subjected to strain from differential settling or being exposed to leachate. HDPE and LLDPE are ideally suited for disposal facility liner and final cover systems. For this project, LLDPE will be used in the final cover system and HDPE will be used in the liner system.

10.1.4 SUBMITTALS

Submit the results of the manufacturer's quality control testing in accordance with the test requirements outlined in Table 5A and 5B as applicable to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the liner material to the job site.

Submit the results of all required prequalification testing as required in Section 3, including interface friction testing, to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the geomembrane material to the job site.

Upon delivery at the site, submit all roll identification and quality control certificates issued by the manufacturer to the on-site CQA consultant representative. Rolls without proper identification will be rejected.

Submit the results of the quality control testing performed during the deployment of the geomembrane. This will include at a minimum:

- Trial welds
- Panel placement logs
- Panel seaming logs
- Non-destructive test results
- Destructive test results
- Repair logs

Submit the Leak Location Survey Work Plan to the Project Engineer for approval prior to commencement of the leak location survey.

10.1.5 Quality Assurance

Prior to delivery of the rolls of geomembrane, the CQA Consultant will verify that representative samples are removed and forwarded to a qualified testing laboratory for testing to verify conformance with the test methods and values presented in Table 6A and 6B as applicable.

10.1.6 Delivery Storage and Handling

Handle the geosynthetic materials with due care and utilize handling equipment on-site that poses minimal risk of damage to the material.

10.2 Products

10.2.1 Material Requirements

FML Barrier Liner for the Disposal Facility Liner: Utilize 60-mil textured HDPE geomembrane for the disposal facility liner system containing no fillers or extenders. The minimum acceptable physical, mechanical, and hydraulic properties of the HDPE-manufactured sheet are outlined in GRI-GM17 and in Table 6A. In the event of conflict, the properties listed in Table 6A will govern. In particular, this project requires a 60-mil minimum thickness HDPE geomembrane.

FML Barrier Liner for the Final Cover: Utilize 40-mil textured LLDPE geomembrane for the final cover system containing no fillers or extenders. The minimum acceptable physical, mechanical, and hydraulic properties of the LLDPE-manufactured sheet are outlined in GRI-GM17 and in Table 6B.

It is the responsibility of the Contractor to verify that the most current version of GRI-GM17 is adhered to.

10.2.2 SOURCE QUALITY CONTROL

Quality Control testing for the geomembrane will be performed by the manufacturer on the representative samples of the proposed material to demonstrate and verify the materials meet minimum requirements outlined in GRI-GM17.

10.3 EXECUTION

10.3.1 CONSTRUCTION AND INSTALLATION

A daily Prepared Subgrade Acceptance form is to be signed and submitted by the installer to the Project Engineer.

Install the geomembrane above a stable subgrade after receipt of approval of the subgrade from the CQA Consultant.

Complete all geosynthetics field installation in accordance with the manufacturer's recommended installation procedures and the construction specifications as outlined in this Section and to the lines and grades shown on the construction drawings.

Ensure the method and equipment used to unroll the panels has the following characteristics:

- Does not cause scratches or crimps in the geomembrane and does not damage the supporting soil
- Minimizes wrinkles and has adequate temporary anchors (e.g., sand bags, tires) to prevent wind damage

For fusion seaming, a rub sheet may be required directly below each overlap of geomembrane to be seamed in order to prevent any moisture build-up between the sheets.

For extrusion seaming, geomembrane is to be cleaned using a disc grinder or equivalent prior to seaming.

Record all construction details for all deployed geomembrane on individual forms acceptable to the Project Engineer. This includes at a minimum:

- Panel placement logs
- Panel seaming logs
- Repair logs
- Non-destructive and destructive test result logs

Prepare panel layout drawings (field sketches) of the deployed and tested geomembrane for review by the Project Engineer.

Complete the installation such that the geomembrane is smooth, without wrinkles, tears, or holes, and covers the total surface of the disposal facility liner and/or cap system. Do not leave tools, debris, or surplus materials on the surface.

Trial Welds

Verify seaming conditions and techniques are adequate by performing daily trial welds on representative pieces of geomembrane in accordance with the following procedures for all combinations of seamer and seaming equipment:

- Perform trial welds once in the morning and once in the afternoon, when operator/machine combinations change, and when an apparatus is turned off and restarted
- Perform additional trial welds when the liner temperature changes by 36°F or more since the previous trial weld was performed

A passing trial seam must be made at the frequency noted above for each seaming device and technician prior to performing production seaming.

Seam Geometry

Seam in accordance with the following specifications:

- Orient seams parallel to the line of maximum slope, i.e., oriented along, not across, the slope
- Minimize the number of seams in corners and odd-shaped geometric locations
- Where horizontal seams are necessary and acceptable to the Project Engineer or CQA Inspector, stagger seams a minimum distance of 10-feet between adjacent seams
- Do not allow horizontal seams to be within 5-feet of the toe of slopes or areas of potential stress concentration unless otherwise approved by the Project Engineer

Ambient Temperatures and Weather Considerations

Do not deploy geomembrane during any precipitation, fog, snow, in areas of ponded water, or in the presence of excessive winds.

Measure sheet temperature prior to seaming by placing a thermometer on the surface of the sheet.

Do not perform seaming where measured surface temperatures are below 32°F or above 104°F for extrusion welding and 140°F for fusion welding. Deviations from these temperature criteria may only occur when authorized by the Owner and with the concurrence of the Project Engineer.

10.3.2 Field Quality Control

During deployment and seaming of the geomembrane, perform the following quality control tests and record the results for review and concurrence by the Project Engineer:

- Trial welds

- Non-destructive test results
- Destructive test results
- Repair logs

Perform non-destructive testing on 100 percent of field seams over their full length using a vacuum test unit (for extrusion seams only), air pressure test, or other acceptable method in accordance with industry accepted standards and in accordance with manufacturer's recommendations.

Complete non-destructive testing of the seams as the seaming work progresses and any required repairs in accordance with industry standards and in accordance with manufacturer's recommendations.

Ensure all field testing equipment is calibrated in accordance with Table 12.

Air Channel Testing

Unless otherwise specified, air pressure testing of the seamed channel will include inflating the test channel, closing the valve, and observing initial pressure after approximate air temperature and pressure have stabilized. The initial pressure will be set appropriately as indicated in Tables 7A and 7B, and the test will last for 5 minutes after reading the initial test pressure. If pressure loss exceeds the allowable specification in the Tables 7A and 7B or if the pressure does not stabilize, locate the faulty area and repair as needed. Flap welding is not an acceptable repair for a failing air channel test. For passing tests, at the end of the 5-minute period, cut the far end of the seam and note the resultant pressure drop.

Vacuum Box Testing

Unless otherwise specified, vacuum testing will be required on all extrusion welded seams. To vacuum test, turn on the vacuum pump to reduce the vacuum box to approximately 5 psi. They will apply liquid soap and water solution to the area to be tested, place the vacuum box over the area to be tested and apply sufficient downward pressure to "seat" the seal strip against the liner. Once a tight seal is created, observe the seam through the window for a period of not less than 10 seconds. If no bubbles appear after 10 seconds, proceed to the next segment of seam to be tested. Mark and repair areas failing the test with a cap strip or other acceptable method.

10.3.3 Field Quality Assurance

The CQA Inspector and Project Engineer will confirm the roll identification corresponds to quality control certificates issued by the manufacturer.

The CQA Inspector will document observations during installation including damage, seaming logs, repair logs, test results, and conformance to specifications. Other items that will be noted include:

- The method and equipment used to unroll the panels does not cause scratches or crimps in the geomembrane and does not damage the supporting soil
- The method used to place the panels for minimization of wrinkles and temporary loading utilized to prevent wind damage

All field testing will be performed in the manner and at the frequency identified in Tables 7A and 7B.

Destructive Testing

Unless an alternative frequency is indicated, the CQA Consultant will choose locations for cutting samples by the Contractor for all destructive seam tests at a frequency no less than one per every 500-feet of seam completed by a particular seamer/apparatus combination.

Cut the samples as the seaming progresses in order to have passing test results before the geomembrane is covered by the overlying materials. The CQA Consultant will:

- Assign a number to each sample, and mark it accordingly
- Observe sample cutting
- Record the sample location on the layout drawing
- Record the reason for taking the sample at this location, if not taken due to statistical routine
- Observe and record all test results

The sample will be divided into three specimens: one for field testing by the Contractor, one for independent laboratory peel and shear testing, and one to the Owner for archive storage.

Test the field specimen with a properly calibrated tensiometer for peel and shear and verify it meets the minimum requirements presented in Table 7A and 7B. If any field test sample fails to meet the minimum requirements, the CQA consultant will determine locations for additional destruct samples for testing a minimum of 10-feet on either side of the failed test location continually bounding the failing test(s) until two bounding, passing tests are recorded. Repair the interval between the passing test locations with a cap strip and test via vacuum box.

If the result is passing, then the CQA Consultant will transport an additional specimen for laboratory testing. Peel and shear destructive seam sample testing will be performed with a calibrated tensiometer. At least five specimens will be tested. If a sample fails laboratory testing, then additional samples will be collected in the field similar to the field testing procedures.

Repair all holes in the geomembrane resulting from destructive seam sampling immediately.

Alternative Destructive Testing Sampling Frequency

As the project continues and data is accumulated, the sampling interval may be varied according to the procedure set forth in GRI GM14. Following this procedure will result in three possible situations:

- Good seaming with fewer rejected test results than the preset historic average can result in a sequential increase in the spacing interval, i.e., one per greater than 500-feet.
- Poor seaming with more rejected test results than the preset historic average can result in a sequential decrease in the spacing interval, i.e., one per less than 500-feet.
- Average seaming with approximately the same test results as the preset historic average will result in the spacing interval remaining the same, i.e., one per 500-feet.

Alternative frequency testing will be implemented at the discretion of the Project Engineer.

11.0 GEOCOMPOSITE

11.1 GENERAL

11.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and installation of this material. Geocomposite will serve as a collection and transport layer of infiltrating liquids such as storm water or leachate in the final cover system and leachate collection system, respectively. Geocomposite typically has several QA/QC requirements for the manufacturer, installer, and the CQA inspector.

11.1.2 DESCRIPTION AND DESIGN REQUIREMENTS

The geocomposite is designed to collect and convey storm water that has infiltrated through the final cover system or entered through the waste material and is retained above the FML barrier layer. For the cover system, water that has infiltrated the cap system will be directed to drainage pipes that will convey the collected water to surface water channels. The geocomposite must be designed and perform adequately to prevent the water level on the geomembrane from building up and potentially causing a slope failure.

For the leachate collection system geocomposite, water collected will be conveyed to leachate collection pipes and to a leachate collection sump where the leachate will be pumped out of the disposal facility cell and sent for treatment. The geocomposite must be designed and perform adequately to minimize the head on the liner system.

11.1.3 SUBMITTALS

Submit the results of the manufacturer's quality control testing in accordance with the test requirements outlined in Table 8 to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the geocomposite material to the job site.

Submit the results of all required prequalification testing as required in Section 3, including interface friction testing, to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the geocomposite material to the job site.

Upon delivery at the site, submit all roll identification and quality control certificates issued by the manufacturer to the on-site CQA Consultant. Rolls without proper identification will be rejected.

11.1.4 QUALITY ASSURANCE

Prior to delivery of the rolls of geocomposite, the CQA Consultant will verify that representative samples are removed and forwarded to a qualified testing laboratory for testing to verify conformance with the test methods and values presented in Table 9.

11.1.5 DELIVERY STORAGE AND HANDLING

Handle the geosynthetic materials with due care and utilize handling equipment on-site that poses minimal risk of damage to the material.

11.2 PRODUCTS

11.2.1 MATERIAL REQUIREMENTS

Geocomposite for the Final Cover: Utilize a geocomposite for the final cover system that meets the minimum transmissivity requirements contained in Table 9 and has the following properties:

- Has a minimum thickness of 250 mils or as required to meet the required minimum transmissivity
- Consists of a geonet with geotextile layers factory heat-bonded to both sides

Geocomposite for the leachate collection system: Utilize a geocomposite for the leachate collection system with that meets the minimum transmissivity requirements contained in Table 9 and has the following properties:

- Has a minimum thickness of 250 mils or as required to meet the required minimum transmissivity.
- Consists of a geonet with geotextile layers factory heat-bonded to both sides

Additional minimum acceptable physical, mechanical, and hydraulic properties for the geocomposite-manufactured sheet are contained in Table 9.

11.2.2 SOURCE QUALITY CONTROL

Quality Control testing for the geocomposite will be performed by the manufacturer on the representative samples of the proposed material to demonstrate and verify the materials meet minimum requirements outlined in Table 8.

11.3 EXECUTION

11.3.1 CONSTRUCTION AND INSTALLATION

Install the geocomposite above geomembrane after receipt of approval from the CQA Consultant ensuring the geomembrane repair documentation is complete.

Complete all geosynthetics field installation in accordance with the manufacturer's recommended installation procedures and the construction specifications as outlined in this Section and to the lines and grades shown on the construction drawings.

Secure or seam each component of the geocomposite (geotextile and geonet) to the like component at seams and overlaps.

Overlap adjacent edges of geonet along the length of the geocomposite a minimum of 2-inches. Join these overlaps by tying the geonet cores together with fasteners or polymeric braid. Space these ties every 5-feet along the roll length to form a complete interlocking of the layers.

Shingle adjoining geocomposite rolls (end to end) along the roll width down in the direction of the slope, with the geonet portion of the top geocomposite overlapping the geonet portion of the bottom geocomposite a minimum of 12-inches across the roll width. Secure the geonet with ties every 12-inches across the roll width and every 6-inches in the anchor trench.

Overlap the bottom layer geotextile. Join the top layer of geotextile by heat bonding or sewing. Overlap the geotextiles a minimum of 4-inches prior to heat bonding or sewing. If heat bonding is to be used, care must be taken to avoid burn through of the geotextile. If sewing of geotextile seams is to be used, a flat (prayer) seam, "J" seam, or "butterfly-folded" seam is recommended, and ensure the seam is a two-thread, double-lock stitch, or a double row of single-thread, chain stitch.

Complete the installation such that the geocomposite is smooth, without wrinkles, tears, or holes, and covers the total surface of the disposal facility liner and/or cap system. Do not leave tools, debris, or surplus materials on the surface.

11.3.2 FIELD QUALITY ASSURANCE

Seams for the geocomposite will be 100% visually inspected, including inspection of the geonet seams, inspection of the lower geotextile overlaps, and inspection of the upper geotextile sewing or heat seaming. Inspection reports will be reviewed by the Project Engineer.

12.0 GEOTEXTILE

12.1 GENERAL

12.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and installation of this material. Geotextile will serve as a permeable, separative, and protective fabric layer between soil, aggregate, geosynthetic, and waste materials. Geotextiles typically have several QA/QC requirements for the manufacturer, installer, and the CQA inspector.

12.1.2 DESCRIPTION AND DESIGN REQUIREMENTS

The geotextile is designed to separate dissimilar subsurface elements without retarding drainage through the materials. Geotextile will be used to wrap the aggregate surrounding the leachate collection pipes to prevent the infiltration of fines into the gravel pack. Geotextile may also be employed between subgrade soils and rip rap armoring and other potential uses as may be illustrated in the Engineering Plans.

12.1.3 SUBMITTALS

Submit the results of the manufacturer's quality control testing in accordance with the test requirements outlined in Table 10 to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the geotextile material to the job site.

Submit the results of all required prequalification testing as required in Section 3, to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the geotextile material to the job site.

Upon delivery at the site, submit all roll identification and quality control certificates issued by the manufacturer to the on-site CQA consultant representative. Rolls without proper identification will be rejected.

12.1.4 QUALITY ASSURANCE

Prior to delivery of the rolls of geotextile, the CQA Consultant will verify that representative samples are removed and forwarded to a qualified testing laboratory for testing to verify conformance with the test methods and values presented in Table 11 as applicable.

12.1.5 DELIVERY STORAGE AND HANDLING

Handle the geotextile materials with due care and utilize handling equipment on-site that poses minimal risk of damage to the material.

12.2 PRODUCTS

12.2.1 MATERIAL REQUIREMENTS

Geotextile for Leachate Collection System: Utilize geotextile fabric for the leachate collection system that meets the minimum material properties outlined in Table 11.

12.2.2 SOURCE QUALITY CONTROL

Quality Control testing for the geotextile will be performed by the manufacturer on the representative samples of the proposed material to demonstrate and verify the materials meet minimum requirements outlined in Table 10.

12.3 EXECUTION

12.3.1 CONSTRUCTION AND INSTALLATION

A daily Prepared Subgrade Acceptance form is to be signed and submitted by the installer to the Project Engineer.

Complete all geosynthetics field installation in accordance with the manufacturer's recommended installation procedures and the construction specifications as outlined in this Section and to the lines and grades shown on the construction drawings.

Complete the installation such that the geotextile is smooth, without wrinkles, tears, or holes. Do not leave tools, debris, or surplus materials on the surface. Installation will be performed in such a manner as to minimize damage and will comply with the following:

- After the wrapping has been removed, a geotextile will not be exposed to sunlight for more than the time specified by the manufacturer.
- On slopes, the geotextiles will be securely anchored and then rolled down the slope in such a manner as to continually keep the geotextile panel in tension.
- In the presence of wind, geotextiles will be weighted with sandbags or the equivalent. Sandbags will be installed during the placement and will remain until replaced with the appropriate overlying material.
- Sandbags will be filled with the fine-grained material and must be handled with care to prevent rupture.
- Geotextiles will be kept continually under tension to minimize the presence of wrinkles in the geotextile.
- Geotextiles will be cut using an approved geotextile cutter only (i.e., an upward cutting hook blade). If in place, special care must be taken to protect other materials from damage which could be caused by the cutting of the geotextiles.
- Precaution will be taken to prevent damage to the underlying layers during placement of the geotextile.

- During placement of geotextiles, care will be taken not to entrap stones, excessive dust, or moisture that could damage the geotextile, generate clogging of drains or filters, or hamper subsequent seaming.
- After installation, the entire surface of the geotextile will be examined, and harmful foreign objects, such as needles, will be removed.

Seams and Overlaps

Geotextiles will be continuously sewn using thread, which is as chemically resistant and UV resistant as the geotextile. Thread will be approved by the CQA Consultant and Owner. Alternately, geotextile may be joined by heat bonding. Spot sewing is not permitted, except for repairs. Geotextiles will be overlapped a minimum of 4-inches prior to seaming or heat bonding. If sewing of geotextile seams is to be used, a flat (prayer) seam, "J" seam, or "butterfly-folded" seam is recommended, and ensure the seam is a two-thread, double-lock stitch, or a double row of single-thread, chain stitch.

Horizontal seams on side slopes should be avoided. However, if horizontal seams are necessary, at least 100-feet will be maintained between horizontal seams of adjacent panels.

Complete the installation such that the geotextile is smooth, without wrinkles, tears, or holes. Do not leave tools, debris, or surplus materials on the surface.

Repairs

Any holes or tears in the geotextile will be repaired as follows:

- Holes in the geotextile will be patched with geotextile of the same unit weight;
- Sufficient overlap will be provided to ensure a suitable seam can be produced, that will not come apart and, when used as a filter, will contain soil; and
- Patches will be sewn and not heat bonded without the approval of the Project Manager.

Care will be taken to remove any soil or other material which may have penetrated the torn geotextile.

Placement of Materials on Geotextiles

Material will be placed on the geotextile in a manner that causes no damage to the geotextile or supporting materials. Low ground-pressure equipment will be used to place the overlying material. The equipment will be operated on the full depth of the layer. Equipment used for placing the overlying material will not be driven directly on the geotextile.

12.3.2 Field Quality Assurance

In addition to reviews of QC documentation, the Project Engineer or CQA Inspector will visually inspect installed geotextile for damage and conformance with the Specifications.

The CQA Inspector and Project Engineer will confirm the roll identification corresponds to quality control certificates issued by the manufacturer.

The CQA Inspector will document observations during installation including damage, seaming logs, repair logs, test results, and conformance to specifications.

13.0 HPDE COLLECTION/CONVEYANCE PIPE

13.1 GENERAL

13.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the testing and construction of this material. HDPE pipe and associated fittings will be used to construct the leachate and surface water management systems.

13.1.2 DESCRIPTION AND DESIGN REQUIREMENTS

HDPE is a thermoplastic piping material that is suitable for both gravity and pressure applications.

13.1.3 SUBMITTALS

Submit the results of the manufacturer's quality control testing to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the HDPE pipe and appurtenances to the job site for each batch/lot of pipe.

Submit the results of all required prequalification testing as required in Section 3, to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the piping to the job site.

Submit the results of the required prequalification testing to the CQA representative as summarized in Table 3 for approval prior to obtaining pipe annulus aggregate material for use.

13.1.4 DELIVERY STORAGE AND HANDLING

Handle the HDPE pipe and fittings with due care and utilize handling equipment on-site that poses minimal risk of damage to the material.

13.2 PRODUCTS

13.2.1 MATERIAL REQUIREMENTS

HDPE piping will consist of Standard Dimension Ratio (SDR) pipe, as specified in the project specifications. HDPE pipe fittings will be furnished by the manufacturer of the pipe with which they are used and will conform to the requirements of ASTM D3261 for standard fittings.

Pipe joints will be fusion welded, using only manufacturer-approved methods and equipment, unless noted otherwise in the construction drawings.

Pipe annulus aggregate requirements are contained in Table 3.

13.2.2 Source Quality Control

Quality Control testing for the HDPE pipe will be performed by the manufacturer on the representative samples of the proposed material to demonstrate and verify the materials meet the project specifications.

The CQA Inspector and Project Engineer will confirm the pipe batch/lot identification corresponds to quality control certificates issued by the manufacturer.

13.3 Execution

13.3.1 Construction and Installation

Complete all HDPE pipe installation in accordance with the manufacturer's recommended installation procedures and the construction specifications as outlined in this Section and to the lines and grades shown on the construction drawings.

<u>Leachate Collection Pipe</u>

Leachate collection piping will be perforated as detailed in the construction drawings. The leachate collection piping will be placed at locations shown; bedded in, and surrounded by a pipe annulus aggregate; and then wrapped with a non-woven geotextile. Non-perforated piping will be pressure tested prior to certification by the CQA Consultant

<u>Fusion Process for Joints</u>

Pipe joints will be fusion welded, using only manufacturer-approved methods and equipment, and in conformance with the procedures outlined below, unless otherwise specified.

- Delivered pipes and fittings will be examined to verify that pipes and fittings are not broken, cracked, or contain otherwise damaged or unsatisfactory material. Prior to fusing, verify that the fusion surface area is clean and free of moisture, dust, dirt, debris of any kind, and foreign material.
- Fusion of the HDPE joints will be performed at temperatures above 32°F, unless otherwise authorized in writing by the Project Engineer.

<u>Pressure Testing of Joints</u>

The joints of non-perforated HDPE pipes will be tested using hydrostatic methods. Other non-destructive test methods may be used only when approved by the Owner. Test pipe in-place after installation and prior to backfilling.

Pipe and fused joint leaks identified during testing will be repaired by cutting out the leaking area and refusing the pipe. After all leaks are repaired, a retest will be performed.

The completed system when tested should be in its proper trench location and allowed time to reach constant and/or ambient temperature before initiating the test.

13.3.2 FIELD QUALITY ASSURANCE

During installation of the HDPE pipe, perform the following quality control tests and record the results for review and concurrence by the Project Engineer:

- Non-destructive pressure test results
- Pipe repair logs

The CQA Inspector will document observations during installation including damage, repair logs, test results, and conformance to specifications. The method and equipment used to fuse pipe joints and conduct pressure testing will also be noted.

The CQA Inspector will document the results of pressure testing. Additionally, should a failed test occur, the CQA inspector will document the nature of leaks discovered and the details of their repair.

14.0 SURFACE WATER CONTROLS STRUCTURES

14.1 GENERAL

14.1.1 SUMMARY

The following section discusses the specific QA/QC requirements for the construction of surface water control structures. Surface water control structures will be constructed to manage surface water runoff from the disposal facility.

14.1.2 DESCRIPTION AND DESIGN REQUIREMENTS

Surface water control structures include channels, benches, ditches, letdowns, culverts, and catch basins. These structures are designed to manage surface water at the site. Precast concrete surface water control structures may also be designed to meet specified traffic loading requirements.

14.1.3 SUBMITTALS

Submit the results of the manufacturer's quality control testing to the CQA Consultant for review and verification that the reported test results meet with project specifications at least 14 days prior to delivery of the erosion control matting/turf control matting (ECM/TCM) and culvert pipe to the job site.

Submit the results of the required prequalification testing for riprap material to the CQA representative as summarized in Table 3 prior to obtaining riprap material for use.

Submit shop drawings and/or manufacturer's certification demonstrating the ability of precast concrete surface water control structures to meet traffic loading requirements as required by the project specifications or construction drawings.

14.1.4 DELIVERY STORAGE AND HANDLING

Handle the ECM/TCM, precast concrete structures, and culvert pipe with due care and utilize handling equipment on-site that poses minimal risk of damage to the material.

14.2 PRODUCTS

14.2.1 MATERIAL REQUIREMENTS

Riprap materials will conform to the requirements of Section 709.03 of the TDOT "Standard Specifications for Road and Bridge Construction" for the riprap material classification specified in the project specifications or construction drawings.

14.2.2 SOURCE QUALITY CONTROL

Perform additional field or laboratory tests as often as is necessary to document the riprap material continues to meet the prequalification specifications contained in the Material Requirements section above.

14.3 EXECUTION

14.3.1 CONSTRUCTION AND INSTALLATION

Excavate channels to the lines and grades shown on the construction drawings. Install ECM/TCM in accordance with the manufacturer's recommended installation procedures and the construction specifications.

14.3.2 FIELD QUALITY ASSURANCE

In addition to reviews of QC documentation, the Project Engineer or CQA Inspector will visually inspect installed surface water control structures for damage and conformance with the Specifications.

The Project Engineer of CQA Inspection will verify that proper burial depth and slope is maintained on pipe installed as part of the surface water control structures.

TABLES

Table 1A. Laboratory Test Methods for the Evaluation of Soil and Aggregate

COMMON TEST NAME	PARAMETER DEFINED	STANDARD METHOD
Soil Classification	Unified Soil Classification System	ASTM D2487
Sieve and Hydrometer Analysis	Particle Size Distribution of Coarse and Fine Grained Soils	ASTM D6913/ASTM D7928
Sieve Analysis for Aggregates	Particle Size Distribution for Aggregates	ASTM D6913
Gradation Analysis	Particle Size Distribution for Riprap	ASTM D5519
Atterberg Limits	Liquid and Plastic Limits, Plasticity Index	ASTM D4318
Standard Proctor	Moisture/Density Relationship Using 5.5 lb (2.46 kg) Rammer and 12 in. (305 mm) Drop	ASTM D698
Modified Proctor	Moisture/Density Relationship Using 10 lb (4.54 kg) Rammer and 18 in. (457.2 mm) Drop	ASTM D1557
Moisture Content	Water to Dry Weight Ratio	ASTM D2216
Permeability: Flex Wall Permeameter	Permeability (Hydraulic Conductivity) on Undisturbed or Remolded Samples of Soil	ASTM D5084
Permeability: Constant Head	Permeability (Hydraulic Conductivity) of Aggregates	ASTM D5084
Carbonate Content	Carbonate Content of Aggregate	ASTM D3042

Notes:

1) Not all tests are required for this site; refer to Tables 3 and 4 in the CQA Plan.
2) Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 1B. Field Test Methods for the Evaluation of Soil and Aggregate

COMMON TEST NAME	PARAMETER DEFINED	STANDARD METHOD
Visual Classification	Maximum Particle Size, General Material Characteristics	ASTM D2488
USDA Classification	Classification of Ability to Support Vegetation	USDA Method
Nuclear Methods	In-Place Density and Moisture Content	ASTM D6938
Moisture Content	In-Place Moisture as Check on Nuclear Densometer Measurements	ASTM D2216
Sand Cone Density	In-Place Density as Check on Nuclear Densometer Measurements	ASTM D1556
Drive Tube Sample	In-Place Density as Check on Nuclear Densometer Measurements	ASTM D2937
Lift Depth Check	Thickness of Placed Soils or Aggregates	Visual Confirmation

Notes:

1) Not all tests are required for this site; refer to Tables 3 and 4 in the CQA Plan.

2) Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 2. Minimum Requirements and Test Frequencies for Shear Strength Prequalification

Item/Interface Description	Material Preparation	Required Tests	Material Selection	Frequency	Specification [1], [2]			Calculation Basis	Certification
Baseliner Floor System (slopes <10%)									
					Peak Secant Angle (degrees)	Normal Stress (psf)	Peak Shear Stress (psf)		
Compacted Soil Liner/ Geosynthetic Clay Liner	Soil is to be compacted to highest moisture and lowest density expected during construction. All geosynthetics interfaces to be oriented same side to same side as in the field. GCL to be hydrated under light confining pressure prior to testing.	Interface Shear Strength ASTM D5321 over range of normal stresses anticipated	Representative samples of geosynthetics and specific soils/aggregates to be used in construction	Twice prior to initial use. Thereafter, once for each construction event using the worst case interface from baseline testing	22.6	2,000	833	Deep seated translational failure analysis	Results compared to CQA Plan Fig 1 and approved as having met minimum interface strength requirements
Geosynthetic Clay Liner/Textured HDPE						5,000	2,082		
Textured HDPE/ Geocomposite						10,000	4,163		
Geocomposite/ Protective Cover Layer						17,500	7,284		
					Residual Secant Angle (degrees)	Normal Stress (psf)	Residual Shear Stress (psf)		
Compacted Soil Liner / Geosynthetic Clay Liner	Soil is to be compacted to highest moisture and lowest density expected during construction. All geosynthetics interfaces to be oriented same side to same side as in the field. GCL to be hydrated under light confining pressure prior to testing.	Interface Shear Strength ASTM D5321 over range of normal stresses anticipated	Representative samples of geosynthetics and specific soils/aggregates to be used in construction	Twice prior to initial use. Thereafter, once for each construction event using the worst case interface from baseline testing	6.3	2,000	221	Deep seated translational failure analysis	Results compared to CQA Plan Fig 1 and approved as having met minimum interface strength requirements
Geosynthetic Clay Liner/Textured HDPE						5,000	552		
Textured HDPE/ Geocomposite						10,000	1,104		
Geocomposite/ Protective Cover Layer						16,500	1,822		
Baseliner Floor System (10% ≤ slopes ≤33%)									
					Residual Secant Angle (degrees)	Normal Stress (psf)	Residual Shear Stress (psf)		
Compacted Soil Liner / Geosynthetic Clay Liner	Soil is to be compacted to highest moisture and lowest density expected during construction. All geosynthetics interfaces to be oriented same side to same side as in the field. GCL to be hydrated under light confining pressure prior to testing.	Interface Shear Strength ASTM D5321 over range of normal stresses anticipated	Representative samples of geosynthetics and specific soils/aggregates to be used in construction	Twice prior to initial use. Thereafter, once for each construction event using the worst case interface from baseline testing	22.5	250	104	Deep seated translational failure analysis (Permit Part F.4, Table 4, Fig A.1)	Results compared to CQA Plan Fig 1 and approved as having met minimum interface strength requirements
Geosynthetic Clay Liner/Textured HDPE									
Textured HDPE/ Geocomposite									
Geocomposite/ Protective Cover Layer									
					Residual Secant Angle (degrees)	Normal Stress (psf)	Residual Shear Stress (psf)		
Compacted Soil Liner / Geosynthetic Clay Liner	Soil is to be compacted to highest moisture and lowest density expected during construction. All geosynthetics interfaces to be oriented same side to same side as in the field. GCL to be hydrated under light confining pressure prior to testing.	Interface Shear Strength ASTM D5321 over range of normal stresses anticipated	Representative samples of geosynthetics and specific soils/aggregates to be used in construction	Twice prior to initial use. Thereafter, once for each construction event using the worst case interface from baseline testing	19.9	250	90	Deep seated translational failure analysis (Permit Part F.4, Table 4, Fig A.1)	Results compared to CQA Plan Fig 2 and approved as having met minimum interface strength requirements
Geosynthetic Clay Liner/Textured HDPE									
Geosynthetic Clay Liner/Textured HDPE									
Textured HDPE/ Geocomposite									

Table 2. Minimum Requirements and Test Frequencies for Shear Strength Prequalification (Continued)

Item/Interface Description	Material Preparation	Required Tests	Material Selection	Frequency	Specification[(1), (2)]			Basis For Test	Certification
Final Cover System									
					Peak Secant Angle (degrees)	Normal Stress (psf)	Peak Shear Stress (psf)		
Protective Cover Soil/ Gecomposite Drainage Gecomposite Drainage/ Geomembrane Liner Geomembrane Liner/ Intermediate Soil Cover	Soil is to be compacted to highest moisture and lowest density expected during construction. All geosynthetics interfaces to be oriented same side to same side as in the field.	Interface Shear Strength ASTM D5321 at normal force equiv. to final cover load.	Representative samples of geosynthetics and specific soils/aggregates to be used in construction	Twice prior to initial use. Thereafter, once for each construction event using the worst case interface from baseline testing	25.8	230	111	Veneer translational failure analysis of final cover slopes	Results evaluated and approved as having met applicable specifications established by the slope stability analysis
					Residual Secant Angle (degrees)	Normal Stress (psf)	Residual Shear Stress (psf)		
Protective Cover Soil/ Gecomposite Drainage Gecomposite Drainage/ Geomembrane Liner Geomembrane Liner/ Intermediate Soil Cover	Soil is to be compacted to highest moisture and lowest density expected during construction. All geosynthetics interfaces to be oriented same side to same side as in the field.	Interface Shear Strength ASTM D5321 at normal force equiv. to final cover load.	Representative samples of geosynthetics and specific soils/aggregates to be used in construction	Twice prior to initial use. Thereafter, once for each construction event using the worst case interface from baseline testing	21	230	88	Veneer translational failure analysis of final cover slopes	Results evaluated and approved as having met applicable specifications established by the slope stability analysis

(1) In situations where a conformance test results falls below the specified minimum, review Figures 1 through 3 following these tables that depict acceptable combinations ϕ and c where c is greater than 0 for each loading scenario. In addition, the particular slope stability calculation can be repeated to determine if additional non-linear analyses and/or reasonable changes in assumptions show the material to be acceptable.

(2) Minimum peak shear strengths are provided in both friction angle and shear stress at specified normal loads. Shear stress (τ) is calculated using the equation $\tau = c + \sigma\tan\phi$ where c = cohesion or adhesion and σ = normal force. Exceeding either the required friction angle with cohesion/adhesion = 0 or the minimum shear stress at the required normal loads is acceptable.

(3) The strength of these materials has been conservatively assumed in the slope stability analysis. These materials should be tested once prior to initial construction to verify the minimum assumed strength parameters are exceeded. Subsequent construction projects should have a geotechnical engineer verify the material characteristics are similar to those tested and verified as passing results. If different materials or a different borrow source are utilized, additional testing should be considered.

(4) The GCL shall be formulated and manufactured by placing bentonite between a scrim-nonwoven carrier geotextile and a nonwoven cap geotextile which are needle punched together to provide internal shear reinforcement. The carrier geotextile shall be placed in contact with the compacted soil liner during construction and interface testing.

Table 2. Minimum Requirements and Test Frequencies for Shear Strength Prequalification (Continued)

Item/Interface Description	Material Preparation	Required Tests	Material Selection	Frequency	Specification[1], [2]			Basis For Test	Certification
Compacted Soil Liner									
					Peak Friction Angle (degrees)/ Cohesion	Normal Stress (psf)	Peak Shear Stress (psf)		
Compacted Soil Liner [3]	Soil at highest moisture and lowest density expected during construction	Consolidated-Undrained Triaxial Compression Test (ASTM D4767)	(1) Representative sample of soil to be used in construction	Whenever there is a change in material	27 / 0	2,000	1,019	Deep seated translational failure analysis of interim waste slopes	Results evaluated and approved as having met applicable specifications established by the slope stability analysis
						5,000	2547		
						10,000	5,095		
						20,000	10,190		
						500	282		
						1000	464		
Protective Cover Soil									
					Peak Friction Angle (degrees)/ Cohesion	Normal Stress (psf)	Peak Shear Stress (psf)		
Protective Cover Layer [3]	Soil at highest moisture and lowest density expected during construction	Direct Shear Test (ASTM D3080)	(1) Representative sample of protective soil to be used in construction	Whenever there is a change in material	25 / 100	150	170	Veneer translational failure analysis of final cover slopes	Results evaluated and approved as having met applicable specifications established by the slope stability analysis
						300	240		
						600	380		

(1) In situations where a conformance test results falls below the specified minimum, review Figures 1 through 3 following these tables that depict acceptable combinations ϕ and c where c is greater than 0 for each loading scenario. In addition, the particular slope stability calculation can be repeated to determine if additional non-linear analyses and/or reasonable changes in assumptions show the material to be acceptable.

(2) Minimum peak shear strengths are provided in both friction angle and shear stress at specified normal loads. Shear stress (τ) is calculated using the equation $\tau = c + \sigma \tan\phi$ where c = cohesion or adhesion and σ = normal force. Exceeding either the required friction angle with cohesion/adhesion = 0 or the minimum shear stress at the required normal loads is acceptable.

(3) The strength of these materials has been conservatively assumed in the slope stability analysis. These materials should be tested once prior to initial construction to verify the minimum assumed strength parameters are exceeded. Subsequent construction projects should have a geotechnical engineer verify the material characteristics are similar to those tested and verified as passing results. If different materials or a different borrow source are utilized, additional testing should be considered

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 3. Minimum Requirements and Test Frequencies for Soil and Aggregate Materials

Component	Required Test	Minimum Frequency	Sample Size(1)	Acceptance Criteria
Subgrade/Structural Fill Prequalification Conformance Testing (3)	Visual Observation	As required	N/A	Substantially free of debris, large rocks, plant materials, or other deleterious material.
	Sieve Analysis (ASTM D6913)	1 per source & 1 per 5,000 yd^3	5-10 lb	Max. 4 in. particle size
	Standard Proctor (ASTM D698)	1 per source & 1 per 10,000 yd^3	50-100 lb	Determination of window of acceptable moisture content given required dry density. Maximum dry unit weight greater than 90 lb/ft3
Subgrade/Structural Fill Performance Testing (4)	Visual Observation	As required	N/A	Final surface: firm, smooth, and uniform
	Lift Depth Check	As required	N/A	6 to 8 in compacted lift
	Nuclear Densometer In-place Density and Moisture Content (ASTM D6938)	1 per 100 ft grid per lift	N/A	> 95% Standard Proctor maximum dry density, Moisture content -4% +4% of optimum
	Proofroll	Continuous or as directed by the Engineer	Varies	No excessive pumping or rutting of subgrade
Geologic Buffer Prequalification Conformance Testing(3)	Visual Observation	As required	N/A	Substantially free of debris, large rocks, plant materials, or other deleterious material
	Sieve Analysis (ASTM D6913)	1 per source & 1 per 5,000 yd^3	5-10 lb	4-inch max. particle size, 2 inch max. particle size for lift in contact with geosynthetics
	Atterberg Limits (ASTM D4318)	1 per source & 1 per 5,000 yd^3	5-10 lb	Plasticity Index: 10 or more2
	Standard Proctor (ASTM D698)	1 per source & 1 per 5,000 yd^3	50-100 lb	Determination of window of acceptable moisture content given required dry density
	Moisture Content (ASTM D2216)	1 per 5,000 yd^3	Varies	Determine if adequate moisture is present prior to compaction
	Flexible Wall Permeability (remolded) (ASTM D5084)	1 per 10,000 yd^3	50 lb	≤1 x 10^{-5} cm/sec; Field test to establish field compaction specification; Certification Engineer to use approved borrow source specification
	Unified Soil Classification (ASTM D2487)	1 per source & 1 per 5,000 yd^3	5-10 lb	SC, SM, GC, CL, CH, or CL-ML
Geologic Buffer Performance Testing (4)	Visual Observation	As required	N/A	Final surface: firm, smooth, and uniform. Perform lift depth check
	Nuclear Densometer In-place Density and Moisture Content (ASTM D 6938)	5 per acre per lift	N/A	≥ 95% Standard Proctor maximum dry density or as required to achieve the required permeability.
	Lift Depth Check	As required	N/A	6 to 8 in. compacted lift. Total 5-ft thickness to be verified by as-built survey at 100-foot maximum grid interval
Compacted Soil Liner Prequalification Conformance Testing(3)	Visual Observation	As required	N/A	Substantially free of debris, large rocks, plant materials, or other deleterious material
	Sieve Analysis (ASTM C136)	1 per source & 1 per 5,000 yd^3	5-10 lb	4-inch max. particle size, 2 inch max. particle size for lift in contact with geosynthetics
	Atterberg Limits (ASTM D4318)	1 per source & 1 per 5,000 yd^3	5-10 lb	Plasticity Index: 10 or more2
	Standard Proctor (ASTM D698)	1 per source & 1 per 5,000 yd^3	50-100 lb	Determination of window of acceptable moisture content given required dry density
	Moisture Content (ASTM D2216)	1 per 5,000 yd^3	Varies	Determine if adequate moisture is present prior to compaction
	Flexible Wall Permeability (remolded) (ASTM D5084)	1 per 10,000 yd^3	50 lb	≤1 x 10^{-6} cm/sec; Field test to establish field compaction specification; Certification Engineer to use approved borrow source specification
	Unified Soil Classification (ASTM D2487)	1 per source & 1 per 5,000 yd^3	5-10 lb	SC, SM, GC, CL, CH, or CL-ML
	Visual Observation	As required	N/A	Final surface: firm, smooth, and uniform. Perform lift depth check
Compacted Soil Liner Performance Testing (4)	Nuclear Densometer In-place Density and Moisture Content (ASTM D 6938)	5 per acre per lift	N/A	≥ 95% Standard Proctor maximum dry density or as required to achieve the required permeability.

	Lift Depth Check	As required	N/A	6 to 8 in. compacted lift. Total 2-ft thickness to be verified by as-built survey at 100-foot maximum grid interval

Table 3. Minimum Requirements and Test Frequencies for Soil and Aggregate Materials (Continued)

Component	Required Test	Minimum Frequency	Sample Size(1)	Acceptance Criteria
Pipe Annulus Aggregate Prequalification Conformance Testing (3	Sieve Analysis (ASTM D6913)	1 per source & 1 per 5,000 yd^3	5-10 lb	Meet Specifications for TDOT No. 57 gravel 100% passing 1-1/2 in. sieve; 95%-100% passing 1 in. sieve; 25% -60% passing 1/2 in sieve; 0%-10% passing #4 sieve; 0-5% passing #8 sieve.
Pipe Annulus Aggregate Performance Testing (4)	Visual Observation	As required	N/A	Pipe satisfactorily supported by aggregate bedding haunched to pipe springline prior to backfill with aggregate.
Final Cover Protective Soil Prequalification Conformance Testing (3)	Visual Observation	As required	N/A	Substantially free of debris, large rocks, plant materials, or other deleterious material. Must not pump or rut excessively.
	Sieve Analysis (ASTM D6913)	1 per source & 1 per 5,000 yd^3	5-10 lb	Max. 1 in. particle size
Final Cover Protective Soil Performance Testing (4)	Visual Observation	As required	N/A	Final surface: firm, smooth, and uniform
	Lift Depth Check	As required	N/A	12 or 18 in. compacted lift. 1.5 ft total thickness verified by as-built survey at 100-foot maximum grid interval
Protective Cover Layer (sand) Prequalification Conformance Testing (3)	Sieve Analysis (ASTM D6913)	1 per source & 1 per 5,000 yd^3	50 lb	Acceptable gradation curve approved by project engineer
	Constant Head Permeability	1 per source & 1 per 5,000 yd^3	50 lb	≥ 4.1×10^{-3} cm/sec or alternatively approved by project engineer
Protective Cover Layer (sand) Performance Testing (4)	Lift Depth Check	As required	N/A	12-inch minimum lift. 1-ft total thickness verified by as-built survey at 100-foot maximum grid interval
Riprap Prequalification Conformance Testing (3)	Gradation Analysis (ASTM D5519)	1 per source per riprap gradation	10 tons	Meets Specifications for TDOT “Standard Specifications for Road and Bridge Construction” Section 709.03

(1) In general, where the symbol "N/A" (not applicable) is used, the test is performed on in-place materials.
(2) Minor variations will be allowed in acceptance criteria for geologic buffer in order to maintain permeability less than 1×10^{-5} cm/sec. Under no circumstances will acceptance criteria be enforced which result in permeability greater than 1×10^{-5} cm/sec
(3) Conformance testing is performed on borrow sources and placed material to ensure the minimum required values are met and the material remains consistent.
(4) Performance testing is performed on materials after placement is complete to ensure that the lift or layer meets design requirements.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 4. Minimum Material Testing Requirements for Reinforced Geosynthetic Clay Liner

Properties	Qualifiers	Units	Specified [1] Property Values	Test Method	Manufacturer's Testing Frequency	Conformance Testing Frequency
Bentonite Mass/Area[2]	MARV	lb/ft^2	0.75	ASTM D 5993	1 per 100,000 SF	1 per 100,000 SF
Bentonite Moisture Content	Maximum	%	35	ASTM D 4643	1 per 100,000 SF	NR
Bentonite Fluid Loss	Maximum	ml	18	ASTM D 5891	1 per 100,000 SF	NR
Bentonite Free Swell	Maximum	ml/2g	24	ASTM D 5890	1 per 100,000 SF	NR
Index Flux	Maximum	$m^3/m^2/s$	$1x10^{-8}$	ASTM D 5887	1 per 100,000 SF	NR
Average Peel Strength	MARV	lb/in	12	ASTM D 6496	1 per 100,000 SF	1 per 100,000 SF
Grab Strength	MARV	lb	60[5]	ASTM D 4632	1 per 100,000 SF	1 per 100,000 SF
Polymer Composition (Geotextile)	Minimum	%	95 polyester or polypropylene	-	Certify	1 per 100,000 SF
Permeability	Minimum	cm/s	$5x10^{-9}$ [6]	ASTM D 5887	1 per 100,000 SF	1 per 100,000 SF
Interface/Internal Shear Strength	Minimum	lb/ft^2	See Table 2 of the CQA Plan[4]	ASTM D 5321	NR[3]	Once per construction season or at material change

(1) MARV = minimum average roll value

(2) Measured at a moisture content of 0 percent

(3) NR = Not Required

(4) Tests will be performed on the entire liner system cross-section and over the range of normal stresses planned for application during shearing as discussed in the applicable section of the QA/QC Plan.

(5) ASTM D 4632 may be modified to use a 4-in wide grip. The results shall be the maximum peak of five specimens averaged in machine direction.

(6) A polymer-amended GCL is required for the project. All GCLs proposed shall undergo site-specific leachate compatibility testing to pre-qualify alternate GCL materials for use.

(7) The GCL shall be formulated and manufactured by placing bentonite between a scrim-nonwoven carrier geotextile and a nonwoven cap geotextile which are needle punched together to provide internal shear reinforcement. The carrier geotextile shall be placed in contact with the compacted soil liner during construction and interface testing (5) Or as required to achieve the required internal shear strength. See Section 4 of the CQA Plan.

Take conformance samples across the entire width of the roll and do not include the first 3 lineal ft. Unless otherwise specified, samples are to be 3 ft long by the roll width. If applicable, mark the machine direction on the samples with an arrow.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 5A. Required 60 Mil Textured HDPE Geomembrane Properties – Manufacturing Quality Control

Material Property[3]	Value	Units	Test Method	Frequency[4]
Resin – Melt Flow Index	≤1.0	g/10 min.	ASTM D1238	50,000 SF
Thickness	60	mil	ASTM D 5994	Per roll
Asperity Height[1][4] (min. avg.)	16	mil	GRI GM12	50,000 SF
Specific Gravity (min.)	0.940	gm/cm^3	ASTM D 792 or ASTM D 1505	50,000 SF
Tensile Properties (each direction)			ASTM D6693 Type IV	
1. Tensile Strength at Break (min. avg.)	90	lb/in.		50,000 SF
2. Elongation at Break (min. avg.)	100	percent		50,000 SF
Tear Resistance (min. avg.)	42	lbs	ASTM D1004, Die C	50,000 SF
Puncture Resistance (min. avg.)	90	lbs	ASTM D 4833	50,000 SF
Carbon Black Content	2-3	percent	ASTM D1603	50,000 SF
Carbon Black Dispersion[2]	Category 1 or 2	Rating	ASTM D5596	50,000 SF
Oxidative Induction Time (min. average)	400	Minutes	ASTM D5885	Per formulation
UV Resistance	50	Percent	ASTM D5885	Per formulation

(1) Or as required to meet the interface shear requirements for the project. See Section 4 of the CQA Plan. No visible variation across the width of the roll will be allowed.

(2) Carbon dispersion for 10 different views: 9 of 10 views will be Category 1 or 2 with one view allowed in Category 3.

(3) Manufacturer's Quality Control testing will be performed at a frequency of one test per every 50,000 ft^2 or one test per resin lot, whichever is more frequent. Thickness testing will be performed on each roll.

(4) Asperity Height measurements will be performed on every roll, alternating between measurements for top of sheet and bottom of sheet.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 5B. Required 40 Mil Textured LLDPE Geomembrane Properties – Manufacturing Quality Control

Material Property[3]	Value	Units	Test Method	Frequency[4]
Resin – Melt Flow Index	≤1.0	g/10 min.	ASTM D1238	50,000 SF
Thickness	40	mil	ASTM D 5994	Per roll
Asperity Height[1] [4] (min. avg.)	16	mil	GRI GM12	50,000 SF
Specific Gravity (max.)	0.939	gm/cm^3	ASTM D 792 or ASTM D 1505	50,000 SF
Tensile Properties (each direction)			ASTM D6693 Type IV	
1. Tensile Strength at Break (min. avg.)	60	lb/in.		50,000 SF
2. Elongation at Break (min. avg.)	250	percent		50,000 SF
Tear Resistance (min. avg.)	22	lbs	ASTM D1004, Die C	50,000 SF
Puncture Resistance (min. avg.)	44	lbs	ASTM D 4833	50,000 SF
Carbon Black Content	2-3	percent	ASTM D1603	50,000 SF
Carbon Black Dispersion[2]	Category 1 or 2	Rating	ASTM D5596	50,000 SF
Oxidative Induction Time (min. average)	400	Minutes	ASTM D5885	Per formulation
UV Resistance	35	Percent	ASTM D5885	Per formulation

(1) Or as required to meet the interface shear requirements for the project. See Section 10 of the CQA Plan. Minimum of ten readings must average specified height. No visible variation across the width of the roll will be allowed.

(2) Carbon dispersion for 10 different views: 9 of 10 views will be Category 1 or 2 with one view allowed in Category 3.

(3) Manufacturer's Quality Control testing will be performed at a frequency of one test per every 50,000 ft^2 or one test per resin lot, whichever is more frequent. Thickness testing will be performed on each roll.

(4) Asperity Height measurements will be performed on every roll, alternating between measurements for top of sheet and bottom of sheet.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 6A. Required 60 Mil Textured HDPE Geomembrane Properties – Conformance Testing

Material Property	Value	Units	Test Method	Frequency[4]
Thickness	60	mil	ASTM D 5994	100,000 SF
Specific Gravity (min.)	0.940	gm/cm^3	ASTM D 792 or ASTM D 1505	100,000 SF
Asperity Height[1] (min. avg.)	16	mil	GRI GM12	100,000 SF Top & Bottom
Tensile Strength at Break (min. avg.)	90	lb/in.	ASTM D6693 Type IV	100,000 SF
Elongation at Break (min. avg.)	100	percent	ASTM D6693 Type IV	100,000 SF
Tear Resistance (min. avg.)	42	lbs	ASTM D1004, Die C	100,000 SF
Puncture Resistance (min. avg.)	90	lbs	ASTM D 4833	100,000 SF
Carbon Black Content	2-3	percent	ASTM D1603	100,000 SF
Carbon Black Dispersion[2]	Category 1 or 2	Rating	ASTM D5596	100,000 SF
UV Resistance	50	Percent	ASTM D5885	100,000 SF
Oxidative Induction Time (min. average)	400	Minutes	ASTM D5885	100,000 SF
Interface/Internal Shear Strength[4]	See Section 4 of the CQA Plan	lb/ft^2	ASTM D 5321	100,000 SF

(1) Or as required to meet the interface shear requirements for the project. See Section 4 of the CQA Plan.

(2) Carbon dispersion for 10 different views: 9 of 10 views will be Category 1 or 2 with one view allowed in Category 3.

(3) Conformance testing will be performed by the Certification Engineer at a minimum frequency of one test per 100,000 ft2 or one test per resin lot, whichever is more frequent. Take samples across the entire width of the roll and do not include the first 3 lineal ft. Unless otherwise specified, samples are to be 3 ft long by the roll width. If applicable, mark the machine direction on the samples with an arrow.

(4) Tests will be performed on the entire liner system cross-section and over the range of normal stresses planned for application during shearing as discussed in Section 10 of this QA/QC Plan.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 6B. Required 40 Mil Textured LLDPE Geomembrane Properties – Conformance Testing

Material Property	Value	Units	Test Method	Frequency(4)
Thickness	40	mil	ASTM D 5994	100,000 SF
Specific Gravity (min.)	0.939	gm/cm^3	ASTM D 792 or ASTM D 1505	100,000 SF
Asperity Height(1) (min. avg.)	16	mil	GRI GM12	100,000 SF Top & Bottom
Tensile Strength at Break (min. avg.)	60	lb/in.	ASTM D6693 Type IV	100,000 SF
Elongation at Break (min. avg.)	250	percent	ASTM D6693 Type IV	100,000 SF
Tear Resistance (min. avg.)	22	lbs	ASTM D1004, Die C	100,000 SF
Puncture Resistance (min. avg.)	44	lbs	ASTM D 4833	100,000 SF
Carbon Black Content	2-3	percent	ASTM D1603	100,000 SF
Carbon Black Dispersion(2)	Category 1 or 2	Rating	ASTM D5596	100,000 SF
Interface/Internal Shear Strength(4)	See Section 4 of the CQA Plan	lb/ft^2	ASTM D 5321	100,000 SF

(1) Or as required to meet the interface shear requirements for the project. See Section 10 of the CQA Plan. Minimum of ten readings must average specified height.

(2) Carbon dispersion for 10 different views: 9 of 10 views will be Category 1 or 2 with one view allowed in Category 3.

(3) Conformance testing will be performed by the Certification Engineer at a minimum frequency of one test per 100,000 ft2 or one test per resin lot, whichever is more frequent. Take samples across the entire width of the roll and do not include the first 3 lineal ft. Unless otherwise specified, samples are to be 3 ft long by the roll width. If applicable, mark the machine direction on the samples with an arrow.

(4) Tests will be performed on the entire liner system cross-section and over the range of normal stresses planned for application during shearing as discussed in Section 10 of this QA/QC Plan.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 7A. Required 60 Mil Textured HDPE Seam Properties

Material Property	Value	Units	Test Method
Shear Strength – Fusion and Extrusion(1)	120	lb/in.	ASTM 6392 Strain rate: 2 in./min. 1 in. strip.
Peel Adhesion			ASTM D6392 Strain rate: 2 in./min. 1 in. strip.
Fusion(2)	91	lb/in.	
Extrusion(3)	78	lb/in.	
Air Channel Testing(4)	30	psi	ASTM D5820
Vacuum Box Testing(5)	3	psi	ASTM D5641

(1) For Shear Testing of both fusion and extrusion welds, the strength of 4 out of 5 specimens should meet or exceed the given value. The 5th can be as low as 80% of the tested value. Required laboratory seam testing will be performed by a geosynthetics testing laboratory at a frequency of one test per 500 linear feet of seam constructed for both extrusion and fusion welding equipment unless otherwise directed by the CQA consultant.

(2) For Peel Testing of fusion welds the strength of 4 out of 5 specimens should meet or exceed the given value. The fifth can be as low as 80% of the tested value. All specimens will fail due to film tear bond or with greater than 25% incursion of the weld (peel).

(3) For Peel Testing of extrusion welds, 1 out of 5 specimens may either achieve <52 lb/in. but be ≥41.6 lb/in. or exhibit greater than 25% incursion of the weld (peel). The remaining four specimens must meet the specified strength and have a maximum of 25% incursion of the weld (peel).

(4) For Air Channel Testing of fusion welds, specimen must maintain pressure for 5 minutes with no less than 3 psi pressure drop. Note pressure drop when far seam is cut.

(5) For Vacuum Box Testing of extrusion welds, examine weld for 10 seconds at the required test pressure through the vacuum box window for evidence of leaks.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 7B. Required 40 Mil Textured LLDPE Seam Properties

Material Property	Value	Units	Test Method
Shear Strength – Fusion and Extrusion(1)	60	lb/in.	ASTM 6392 Strain rate: 2 in./min. 1 in. strip.
Peel Adhesion			ASTM D6392 Strain rate: 2 in./min. 1 in. strip.
Fusion(2)	50	lb/in.	
Extrusion(3)	44	lb/in.	
Air Channel Testing(4)	25	psi	ASTM D5820
Vacuum Box Testing(5)	3	psi	ASTM D5641

(1) For Shear Testing of both fusion and extrusion welds, the strength of 4 out of 5 specimens should meet or exceed the given value. The 5th can be as low as 80% of the tested value. Required laboratory seam testing will be performed by a geosynthetics testing laboratory at a frequency of one test per 500 linear feet of seam constructed for both extrusion and fusion welding equipment unless otherwise directed by the CQA consultant.

(2) For Peel Testing of fusion welds the strength of 4 out of 5 specimens should meet or exceed the given value. The fifth can be as low as 80% of the tested value. All specimens will fail due to film tear bond or with greater than 25% incursion of the weld (peel).

(3) For Peel Testing of extrusion welds, 1 out of 5 specimens may either achieve <52 lb/in but be ≥41.6 lb/in. or exhibit greater than 25% incursion of the weld (peel). The remaining four specimens must meet the specified strength and have a maximum of 25% incursion of the weld (peel).

(4) For Air Channel Testing of fusion welds, specimen must maintain pressure for 5 minutes with no less than 3 psi pressure drop. Note pressure drop when far seam is cut.

(5) For Vacuum Box Testing of extrusion welds, examine weld for 10 seconds at the required test pressure through the vacuum box window for evidence of leaks.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 8. Required Geocomposite Properties – Manufacturing Quality Control

Material Property	Value	Units	Test Method	Manufacturer's Frequency
Geonet Component[3]:				
Thickness (min.) Final Cover System	250	mil	ASTM D 5199	50,000 SF
Thickness (min.) Leachate Collection System	250	mil	ASTM D 5199	50,000 SF
Specific Gravity (min. avg.)	0.94	gm/cm^3	ASTM D 792 or ASTM D 1505	50,000 SF
Carbon Black Content	2-3	percent	ASTM D1603	50,000 SF
Tensile Strength, MD (Machine Direction)	45	lb/ft	ASTM D 5035	50,000 SF
Geotextile Component:				
Polymer Composition (min.)	95	% polypropylene or polyester by weight		Certify
Mass per Unit Area (min.)	8	oz/yd^2	ASTM D 5261	50,000 SF
Grab Tensile Strength	≥230	lbs	ASTM D 4632	50,000 SF
Grab Elongation (min. avg.)	50	percent	ASTM D 4632	50,000 SF
Puncture Strength (min.)	120	lbs	ASTM D 4833	100,000 SF
Apparent Opening Size	70 - 120	sieve size	ASTM D 4751	550,000 SF
Water Flow Rate (min.)	110	gpm/ft^2	ASTM D 4491	550,000 SF
Ultraviolet Resistance (min. avg.)	70	percent	ASTM D 4355 (after 500 hours)	1/per lot
Geocomposite:				
Transmissivity at 1,000 psf[1] (min.) Final Cover System	3.75×10^{-4}	m^2/sec	ASTM D 4716	500,000 SF
Transmissivity at 13,500 psf[2] (min.) Leachate Collection System	8.20×10^{-5}	m^2/sec	ASTM D 4716	500,000 SF
Peel Strength (min.)	1	lb/in	GR1 GC-7	500,000 SF

(1) Transmissivity measured using water at 20°C with a gradient of 0.33 and normal stress of up to 1,000 psf using actual boundary conditions for 100 hours or as required to achieve a steady state.

(2) Transmissivity measured using water at 20°C with a gradient of 0.03 and normal stress of 1,000 psf using actual boundary conditions for 100 hours or as required to achieve a steady state.

(3) Based on the angularity of the overlying material, the following requirement for allowable geonet grid opening size will be met in order to protect the liner geomembrane against puncture.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

ANGULARITY OF DRAINAGE LAYER	MAXIMUM ALLOWABLE GEONET GRID OPENING SIZE (IN)
Angular	<0.40
Subangular	<0.56
Rounded	<0.79
Sand/Soil	<0.79

Table 9. Required Geocomposite Properties - Conformance Testing

Material Property	Value	Units	Test Method	Frequency[1]
Geonet Component:				
Thickness (min.) Final Cover System	250	mil	ASTM D5199	100,000 SF
Thickness (min.) Leachate Collection System	250	mil	ASTM D5199	100,000 SF
Resin Density (min. avg.)	0.935	gm/cm^3	ASTM D1505	100,000 SF
Geocomposite:				
Peel Strength (min.)	1	lb/in.	GRI GC-7	100,000 SF
Tensile Strength (min.)	45	lb/in.	ASTM D5035	100,000 SF
Transmissivity at 1,000 psf[2] (min.) Final Cover System	3.75×10^{-4}	m^2/sec	ASTM D4716	100,000 SF
Transmissivity at 13,500 psf[3] (min.) Leachate Collection System	8.20×10^{-5}	m^2/sec	ASTM D4716	100,000 SF

(1) Conformance testing will be performed at a minimum frequency of one test per 100,000 ft^2 or one test per resin lot, whichever is more frequent

(2) Transmissivity measured using water at 20°C with a gradient of 0.33 and normal stress of up to 1,000 psf using actual boundary conditions for 100 hours or as required to achieve a steady state.

(3) Transmissivity measured using water at 20°C with a gradient of 0.03 and normal stress of 13,500 psf using actual boundary conditions for 100 hours or as required to achieve a steady state.

Take samples across the entire width of the roll and do not include the first 3 lineal ft. Unless otherwise specified, samples are to be 3 ft long by the roll width. If applicable, mark the machine direction on the samples with an arrow.

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 10. Required Properties For Nonwoven Geotextiles – Manufacturing Quality Control

Material Property	Value(1)	Units	Test Method	Manufacturer's Frequency
10 oz/sy Nonwoven Geotextile				
Mass/Area (min. avg.)	10	oz/sy	ASTM D 5261	100,000 SF
Permittivity	1.0	sec^{-1}	ASTM D 4491	750,000 SF
Grab Tensile	250	lb	ASTM D 4632	100,000 SF
Grab Elongation	45	percent	ASTM D 4632	100,000 SF
Puncture Resistance	120	lb	ASTM D 4833	100,000 SF
Trapezoidal Tear	100	lb	ASTM D 4533	100,000 SF
Apparent Opening Size	100	U.S. Sieve	ASTM D 4751	750,000 SF
8 oz/sy Nonwoven Geotextile				
Mass/Area (min. avg.)	8	oz/sy	ASTM D 5261	100,000 SF
Permittivity	1.0	sec^{-1}	ASTM D 4491	750,000 SF
Grab Tensile	190	lb	ASTM D 4632	100,000 SF
Grab Elongation	45	percent	ASTM D 4632	100,000 SF
Puncture Resistance	80	lb	ASTM D 4833	100,000 SF
Trapezoidal Tear	75	lb	ASTM D 4533	100,000 SF
Apparent Opening Size	80	U.S. Sieve	ASTM D 4751	750,000 SF
6 oz/sy Nonwoven Geotextile				
Mass/Area (min. avg.)	6	oz/sy	ASTM D 5261	100,000 SF
Permittivity	1.0	sec^{-1}	ASTM D 4491	750,000 SF
Grab Tensile	150	lb	ASTM D 4632	100,000 SF
Grab Elongation	45	percent	ASTM D 4632	100,000 SF
Puncture Resistance	60	lb	ASTM D 4833	100,000 SF
Trapezoidal Tear	60	lb	ASTM D 4533	100,000 SF
Apparent Opening Size	70	U.S. Sieve	ASTM D 4751	750,000 SF

Notes:

(1) Minimum average roll values

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 11. Required Properties For Nonwoven Geotextiles – Conformance Testing

Material Property	Value[1]	Units	Test Method	Frequency
Mass/Area (min. avg.)	6	oz/sy	ASTM D 5261	250,000 SF
Permittivity	1.0	sec^{-1}	ASTM D 4491	1,500,000 SF
Grab Tensile	248	lb	ASTM D 4632	250,000 SF
Grab Elongation	50	percent	ASTM D 4632	250,000 SF
Puncture Resistance	120	lb	ASTM D 4833	250,000 SF
Trapezoidal Tear	90	lb	ASTM D 4533	250,000 SF
Apparent Opening Size	100	U.S. Sieve	ASTM D 4751	1,500,000 SF

Notes:

(1) Minimum average roll values

Latest version of the applicable ASTM International or USDA testing standards will be used when conducting tests.

Table 12. CALIBRATION OF TESTING EQUIPMENT

Equipment	Required Test	Minimum Frequency	Acceptance Criteria
Nuclear Density Gauge	Radioactive Source Wipe Testing and Systems Electronics Check	Annually by Manufacturer or Specialty Testing firm qualified to inspect and calibrate nuclear source equipment	Certificate of Calibration and Safety by Testing Firm
Tensiometer	Tensile strength calibration to standard	Prior to arrival to project site. Tensionmeter to be field verified at the discretion of the Engineer	+/- 3 psi
Air Pressure Gauges	Pressure in psi compared to standard	Prior to arrival to project site or documentation that the product is new	+/- 1 psi
Other	As Determined by the Engineer	As Recommended by the Manufacturer, or Required by State Auditor of Measurement Devices	As Guaranteed by the Manufacturer

Table 13. Reference Table of ASTM or USDA Testing Standards

Standard Title	Designation ID
Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	ASTM D5084 - 16a
Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft3 (600 kN-m/m3))	ASTM D698 - 12e2
Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3))	ASTM D1557 - 12e1
Standard Test Methods for Particle Size Analysis of Natural and Man-Made Riprap Materials	ASTM D5519 - 15
Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	ASTM D4318 - 17
Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass	ASTM D2216 - 10
Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)	ASTM D2487 - 11
Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method	ASTM D2937 - 17e1
Standard Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method	ASTM D1556 / D1556M - 15e1
Standard Test Method for Insoluble Residue in Carbonate Aggregates	ASTM D3042 - 17
Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)	ASTM D2488 - 17
Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)	ASTM D6938 - 17
Standard Specification for Butt Heat Fusion Polyethylene (PE) Plastic Fittings for Polyethylene (PE) Plastic Pipe and Tubing	ASTM D3261 - 16

Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	ASTM D4767 - 11
Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis	ASTM D7928 - 17
Standard Test Method for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis	ASTM D6913M - 17
Standard Test Method for Determination of Water Content of Soil and Rock by Microwave Oven Heating	ASTM D4643 - 17
Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	ASTM D5084 - 16a
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	ASTM D3080 / D3080M - 11
Standard Test Method for Measuring Mass Per Unit of Geosynthetic Clay Liners	ASTM D5993 - 14
Standard Test Method for Fluid Loss of Clay Component of Geosynthetic Clay Liners	ASTM D5891 / D5891M - 02(2016)e1
Standard Test Method for Swell Index of Clay Mineral Component of Geosynthetic Clay Liners	ASTM D5890 - 11
Standard Test Method for Measurement of Index Flux Through Saturated Geosynthetic Clay Liner Specimens Using a Flexible Wall Permeameter	ASTM D5887 / D5887M - 16
Standard Test Method for Determining Average Bonding Peel Strength Between Top and Bottom Layers of Needle-Punched Geosynthetic Clay Liners	ASTM D6496 / D6496M - 04A(2015)e1
Standard Test Method for Grab Breaking Load and Elongation of Geotextiles	ASTM D4632 / D4632M - 15a
Standard Test Method for Determining the Shear Strength of Soil-Geosynthetic and Geosynthetic-Geosynthetic Interfaces by Direct Shear	ASTM D5321 / D5321M - 17

Standard Test Method for Melt Flow Rates of Thermoplastics by Extrusion Plastometer	ASTM D1238 - 13
Standard Test Method for Measuring Core Thickness of Textured Geomembranes	ASTM D5994 / D5994M - 10(2015)e1
Standard Test Methods for Density and Specific Gravity (Relative Density) of Plastics by Displacement	ASTM D792 - 13
Standard Test Method for Density of Plastics by the Density-Gradient Technique	ASTM D1505 - 10
Standard Test Method for Determining Tensile Properties of Nonreinforced Polyethylene and Nonreinforced Flexible Polypropylene Geomembranes	ASTM D6693 / D6693M - 04(2015)e1
Standard Test Method for Tear Resistance (Graves Tear) of Plastic Film and Sheeting	ASTM D1004 - 13
Standard Test Method for Index Puncture Resistance of Geomembranes and Related Products	ASTM D4833 / D4833M - 07(2013)e1
Standard Test Method for Carbon Black Content in Olefin Plastics	ASTM D1603 - 14
Standard Test Method For Microscopic Evaluation of the Dispersion of Carbon Black in Polyolefin Geosynthetics	ASTM D5596 - 03(2016)
Standard Test Method for Oxidative Induction Time of Polyolefin Geosynthetics by High-Pressure Differential Scanning Calorimetry	ASTM D5885 / D5885M - 17
Standard Test Method for Determining the Integrity of Nonreinforced Geomembrane Seams Produced Using Thermo-Fusion Methods	ASTM D6392 - 12
Standard Practice for Pressurized Air Channel Evaluation of Dual Seamed Geomembranes	ASTM D5820 - 95(2011)
Standard Practice for Geomembrane Seam Evaluation by Vacuum Chamber	ASTM D5641 / D5641M - 16

Standard Test Method for Measuring the Nominal Thickness of Geosynthetics	ASTM D5199 - 12
Standard Test Methods for Density and Specific Gravity (Relative Density) of Plastics by Displacement	ASTM D792 - 13
Standard Test Method for Density of Plastics by the Density-Gradient Technique	ASTM D1505 - 10
Standard Test Method for Carbon Black Content in Olefin Plastics	ASTM D1603 - 14
Standard Test Method for Breaking Force and Elongation of Textile Fabrics (Strip Method)	ASTM D5035 - 11(2015
Standard Test Method for Measuring Mass per Unit Area of Geotextiles	ASTM D5261 - 10
Standard Test Method for Grab Breaking Load and Elongation of Geotextiles	ASTM D4632 / D4632M - 15a
Standard Test Method for Index Puncture Resistance of Geomembranes and Related Products	ASTM D4833 / D4833M - 07(2013)e1
Standard Test Methods for Determining Apparent Opening Size of a Geotextile	ASTM D4751 - 16
Standard Test Methods for Water Permeability of Geotextiles by Permittivity	ASTM D4491 / D4491M - 17
Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus	ASTM D4355 / D4355M - 14
Standard Test Method for Determining the (In-plane) Flow Rate per Unit Width and Hydraulic Transmissivity of a Geosynthetic Using a Constant Head	ASTM D4716 / D4716M - 14
Standard Test Method for Measuring Mass per Unit Area of Geotextiles	ASTM D5261 - 10
Standard Test Methods for Water Permeability of Geotextiles by Permittivity	ASTM D4491 / D4491M - 17

Standard Test Method for Grab Breaking Load and Elongation of Geotextiles	ASTM D4632 / D4632M - 15a
Standard Test Method for Index Puncture Resistance of Geomembranes and Related Products	ASTM D4833 / D4833M - 07(2013)e1
Standard Test Method for Trapezoid Tearing Strength of Geotextiles	ASTM D4533 / D4533M - 15
Standard Test Methods for Determining Apparent Opening Size of a Geotextile	ASTM D4751 - 16
Standard Test Method for Measuring Mass per Unit Area of Geotextiles	ASTM D5261 - 10
Standard Test Methods for Water Permeability of Geotextiles by Permittivity	ASTM D4491 / D4491M - 17
Standard Test Method for Grab Breaking Load and Elongation of Geotextiles	ASTM D4632 / D4632M - 15a
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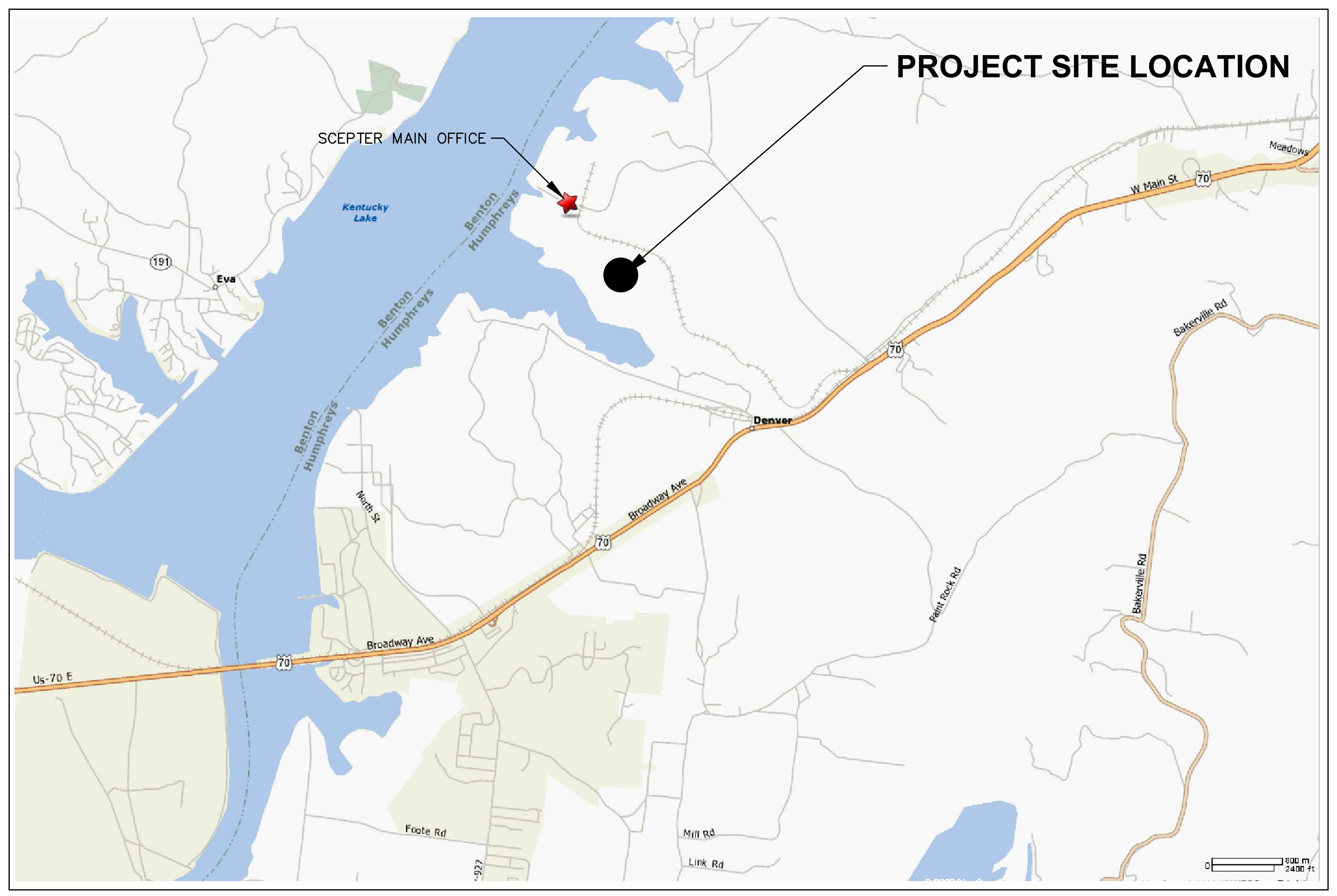
Standard Test Method for Measuring Mass per Unit Area of Geotextiles	ASTM D5261 - 10
Standard Test Methods for Water Permeability of Geotextiles by Permittivity	ASTM D4491 / D4491M - 17

PART E
ENGINEERING PLANS

SCEPTER, INC.
EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

WAVERLY, TENNESSEE
APRIL 05, 2018



SITE VICINITY MAP

0 4800
SCALE 1" = 4800'

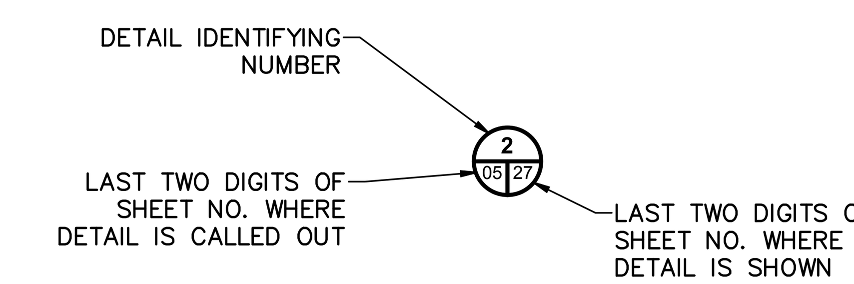
OWNER OF FACILITY LAND(S):

SCEPTER, INC.
1485 SCEPTER LANE
WAVERLY, TN 37185

PREPARED BY:

AECOM
1000 CORPORATE CENTRE DRIVE
SUITE 250
FRANKLIN, TN 37067
PHONE (615) 771-2480
ATTN: MICHAEL MAY, P.E.

INDEX OF DRAWINGS



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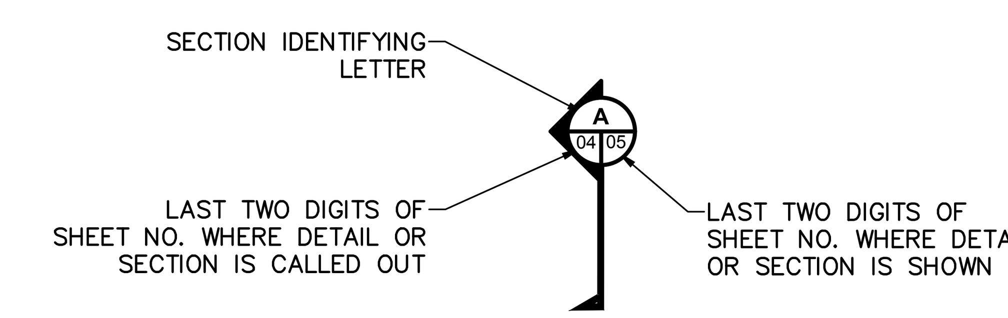

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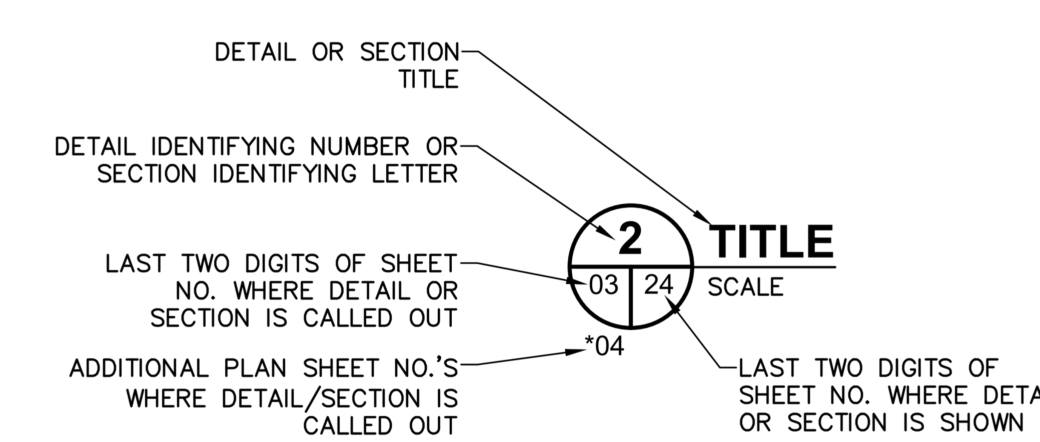


DETAIL OR SECTION, NUMBER OR LETTER & TITLE ON SHEET WHERE DETAIL OR SECTION IS SHOWN

DETAILING & SECTION IDENTIFICATION NOMENCLATURE

ISSUED FOR BIDDING ____ DATE BY

ADDENDUM REVISIONS		
ADDENDUM NO.	ADDENDUM DATE	BY

ISSUED FOR CONSTRUCTION ____ DATE BY

REVISIONS		
NO.	DATE	BY

ISSUED FOR REVIEW ____ DATE BY

REVISIONS			
NO.	COMMENTS	DATE	BY
0	ISSUED FOR PERMITTING	7/1/16	NSP
1	LEACHATE TANK, ROAD SECTION	8/10/16	NSP
2	TDEC PERMIT REVIEW COMMENTS	1/10/17	NSP
3	TDEC PERMIT REVIEW COMMENTS	5/9/17	NSP
4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

RECORD DRAWINGS ____ DATE BY

ISSUED FOR PERMITTING, NOT FOR CONSTRUCTION

MICHAEL S. MAY
REGISTERED ENGINEER
AGRICULTURE COMMERCE
NO. 103104
STATE OF TENNESSEE

DRAWN BY: NSP	DATE 04/05/2018
CHECKED BY: MSM	JOB NO.: 60398526
SCALE AS SHOWN	

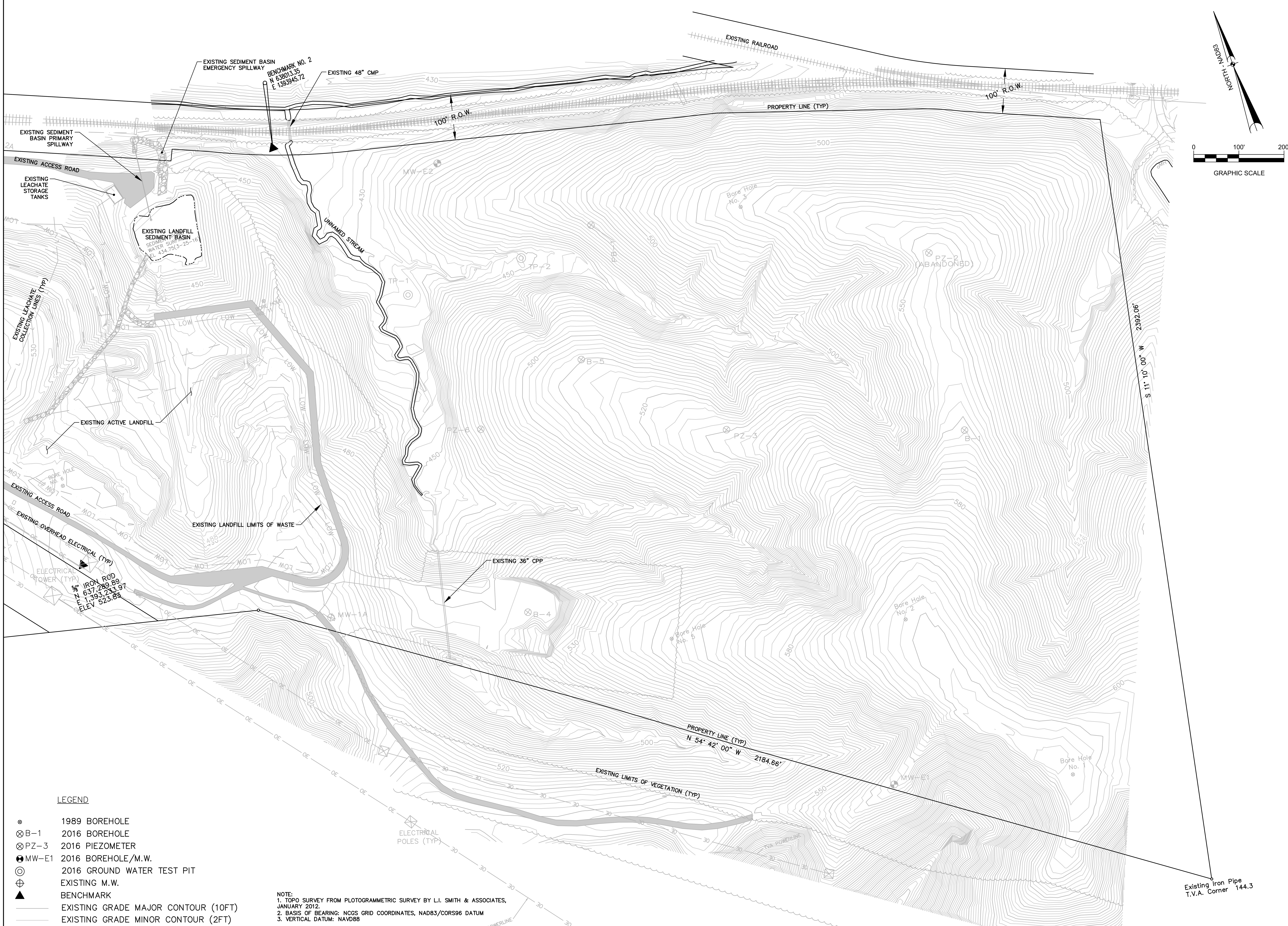
EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

COVER SHEET

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-1



LEGEND

- ⊗ 1989 BOREHOLE
- ⊗B-1 2016 BOREHOLE
- ⊗PZ-3 2016 PIEZOMETER
- MW-E1 2016 BOREHOLE/M.W.
- ◎ 2016 GROUND WATER TEST PIT
- ⊕ EXISTING M.W.
- ▲ BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (10FT)
- EXISTING GRADE MINOR CONTOUR (2FT)

NOTE:
1. TOPO SURVEY FROM PLOTOGRAMMETRIC SURVEY BY L.I. SMITH & ASSOCIATES, JANUARY 2012.
2. BASIS OF BEARING: NCGS GRID COORDINATES, NAD83/CORS96 DATUM
3. VERTICAL DATUM: NAVD88

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ISSUED FOR BIDDING DATE BY

ADDENDUM REVISIONS

ADDENDUM NO.	ADDENDUM DATE	BY

ISSUED FOR CONSTRUCTION DATE BY

REVISIONS

NO.	DATE	BY

ISSUED FOR REVIEW DATE BY

REVISIONS

NO.	COMMENTS	DATE	BY
0	ISSUED FOR PERMITTING	7/1/16	NSP
1	LEACHATE TANK, ROAD SECTION	8/10/16	NSP
2	TDEC PERMIT REVIEW COMMENTS	1/10/17	NSP
3	TDEC PERMIT REVIEW COMMENTS	5/9/17	NSP
4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

RECORD DRAWINGS DATE BY

ISSUED FOR PERMITTING, NOT FOR CONSTRUCTION

MICHAEL S. MAY REGISTERED ENGINEER AGRICULTURE COMMERCE NO. 103104 STATE OF TENNESSEE

DRAWN BY: NSP	DATE 04/05/2018
CHECKED BY: MSM	JOB NO.: 60398526
SCALE AS SHOWN	

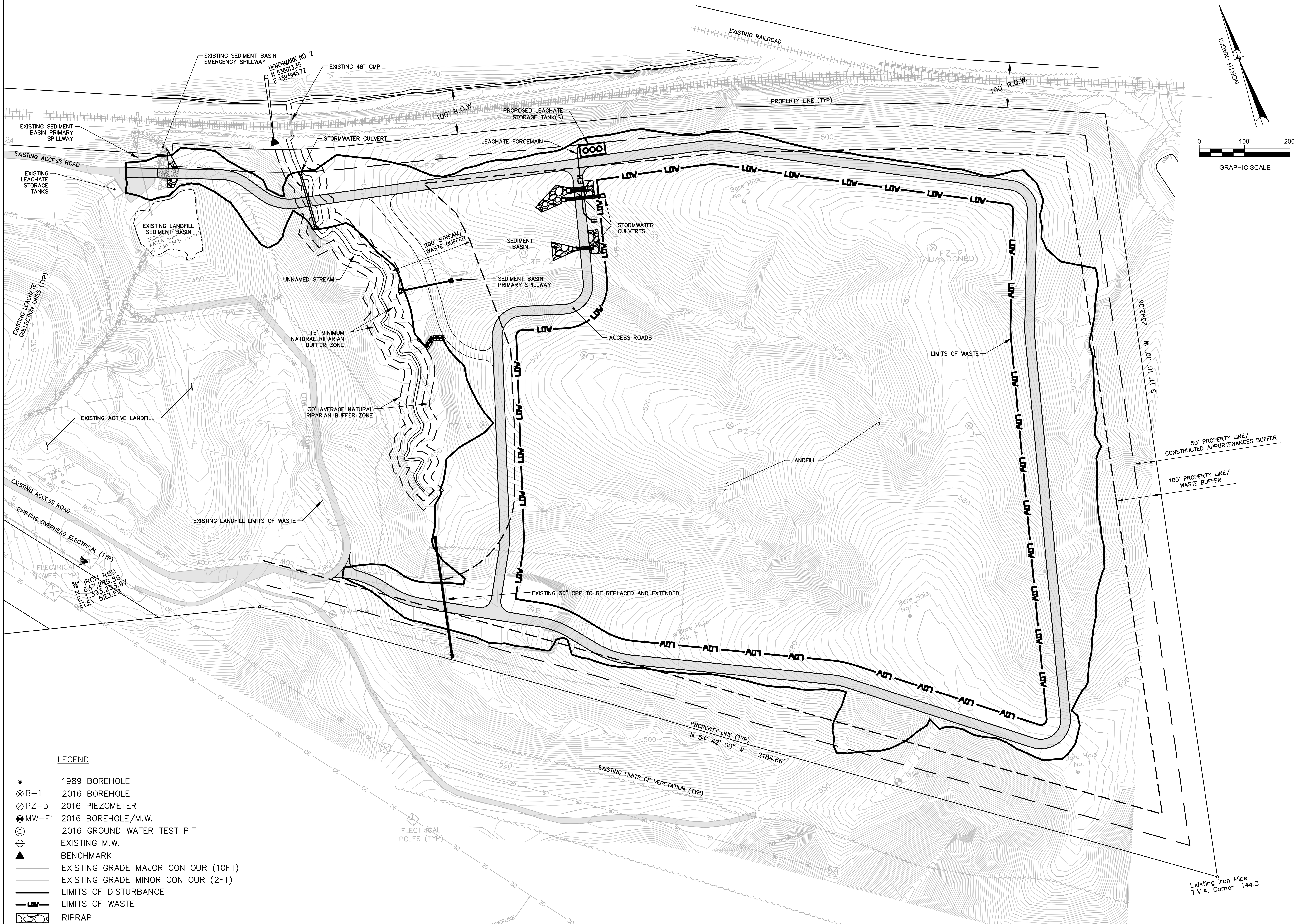
EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

EXISTING CONDITIONS

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-2



LEGEND

- 1989 BOREHOLE
- B-1 2016 BOREHOLE
- PZ-3 2016 PIEZOMETER
- MW-E1 2016 BOREHOLE/M.W.
- 2016 GROUND WATER TEST PIT
- EXISTING M.W.
- BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (10FT)
- EXISTING GRADE MINOR CONTOUR (2FT)
- LIMITS OF DISTURBANCE
- LOW LIMITS OF WASTE
- RIPRAP

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ISSUED FOR BIDDING DATE BY

ADDENDUM REVISIONS		
ADDENDUM NO.	ADDENDUM DATE	BY

ISSUED FOR CONSTRUCTION DATE BY

REVISIONS		
NO.	DATE	BY

ISSUED FOR REVIEW DATE BY

REVISIONS			
NO.	COMMENTS	DATE	BY
0	ISSUED FOR PERMITTING	7/1/16	NSP
1	LEACHATE TANK, ROAD SECTION	8/10/16	NSP
2	TDEC PERMIT REVIEW COMMENTS	1/10/17	NSP
3	TDEC PERMIT REVIEW COMMENTS	5/9/17	NSP
4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

RECORD DRAWINGS DATE BY

ISSUED FOR PERMITTING, NOT FOR CONSTRUCTION

MICHAEL S. MAY REGISTERED ENGINEER AGRICULTURE COMMERCE NO. 103104 STATE OF TENNESSEE

DRAWN BY: NSP	DATE 04/05/2018
CHECKED BY: MSM	JOB NO.: 60398526
SCALE AS SHOWN	

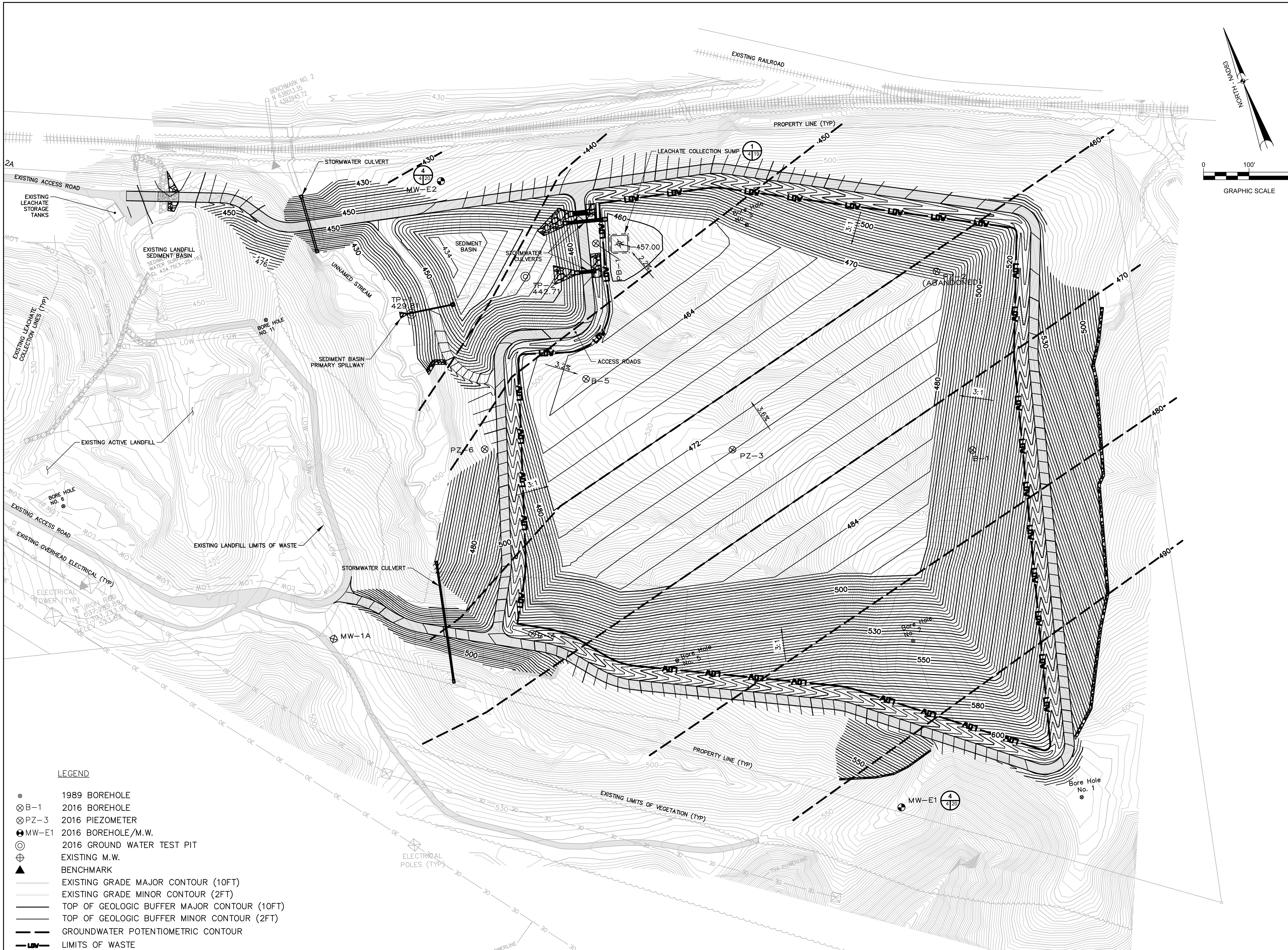
EAST PHASE DISPOSAL FACILITY PERMIT DRAWINGS

SITING CRITERIA BUFFERS/SETBACKS

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-3



LEGEND

- 1989 BOREHOLE
- B-1 2016 BOREHOLE
- PZ-3 2016 PIEZOMETER
- MW-E1 2016 BOREHOLE/M.W.
- 2016 GROUND WATER TEST PIT
- EXISTING M.W.
- BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (10FT)
- EXISTING GRADE MINOR CONTOUR (2FT)
- TOP OF GEOLOGIC BUFFER MAJOR CONTOUR (10FT)
- TOP OF GEOLOGIC BUFFER MINOR CONTOUR (2FT)
- GROUNDWATER POTENTIOMETRIC CONTOUR
- LOW LIMITS OF WASTE

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ISSUED FOR REVIEW DATE BY

REVISIONS

NO.	COMMENTS	DATE	BY
0	ISSUED FOR PERMITTING	7/1/16	NSP
1	LEACHATE TANK, ROAD SECTION	8/10/16	NSP
2	TDEC PERMIT REVIEW COMMENTS	1/10/17	NSP
3	TDEC PERMIT REVIEW COMMENTS	5/9/17	NSP
4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

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MICHAEL S. MAY REGISTERED ENGINEER AGRICULTURE COMMERCE NO. 103104 STATE OF TENNESSEE

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SCALE AS SHOWN	

EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

TOP OF GEOLOGIC BUFFER

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-4

NOTE:
SOIL BORROW FROM WITHIN THE LIMITS OF WASTE AREA.

LEGEND

- ▲ BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (10FT)
- EXISTING GRADE MINOR CONTOUR (2FT)
- TOP OF SOIL LINER MAJOR CONTOUR (10FT)
- TOP OF SOIL LINER MINOR CONTOUR (2FT)
- LOW LIMITS OF WASTE

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NO.	DATE	BY

ISSUED FOR REVIEW ____ DATE ____ BY

REVISIONS

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5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

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MICHAEL S. MAY
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
NO. 103104
STATE OF TENNESSEE

DRAWN BY: NSP	DATE 04/05/2018
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EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

TOP OF COMPACTED SOIL LINER

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-5

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NOTE:
NO FITTINGS GREATER THAN 45 DEGREES SHALL BE USED AS PART OF THE LEACHATE COLLECTION SYSTEM UNLESS NOTED ON THE CONSTRUCTION DOCUMENTS.

LEGEND

▲	BENCHMARK
	EXISTING GRADE MAJOR CONTOUR (10FT)
	EXISTING GRADE MINOR CONTOUR (2FT)
	TOP OF CLAY LINER MAJOR CONTOUR (10FT)
	TOP OF CLAY LINER MINOR CONTOUR (2FT)
LOW	LIMITS OF WASTE
L	LEACHATE COLLECTION PIPES
C/O	LEACHATE COLLECTION PIPE CLEANOUT

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REVISIONS

NO.	DATE	BY

ISSUED FOR REVIEW DATE BY

REVISIONS

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EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

LEACHATE COLLECTION SYSTEM

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-6

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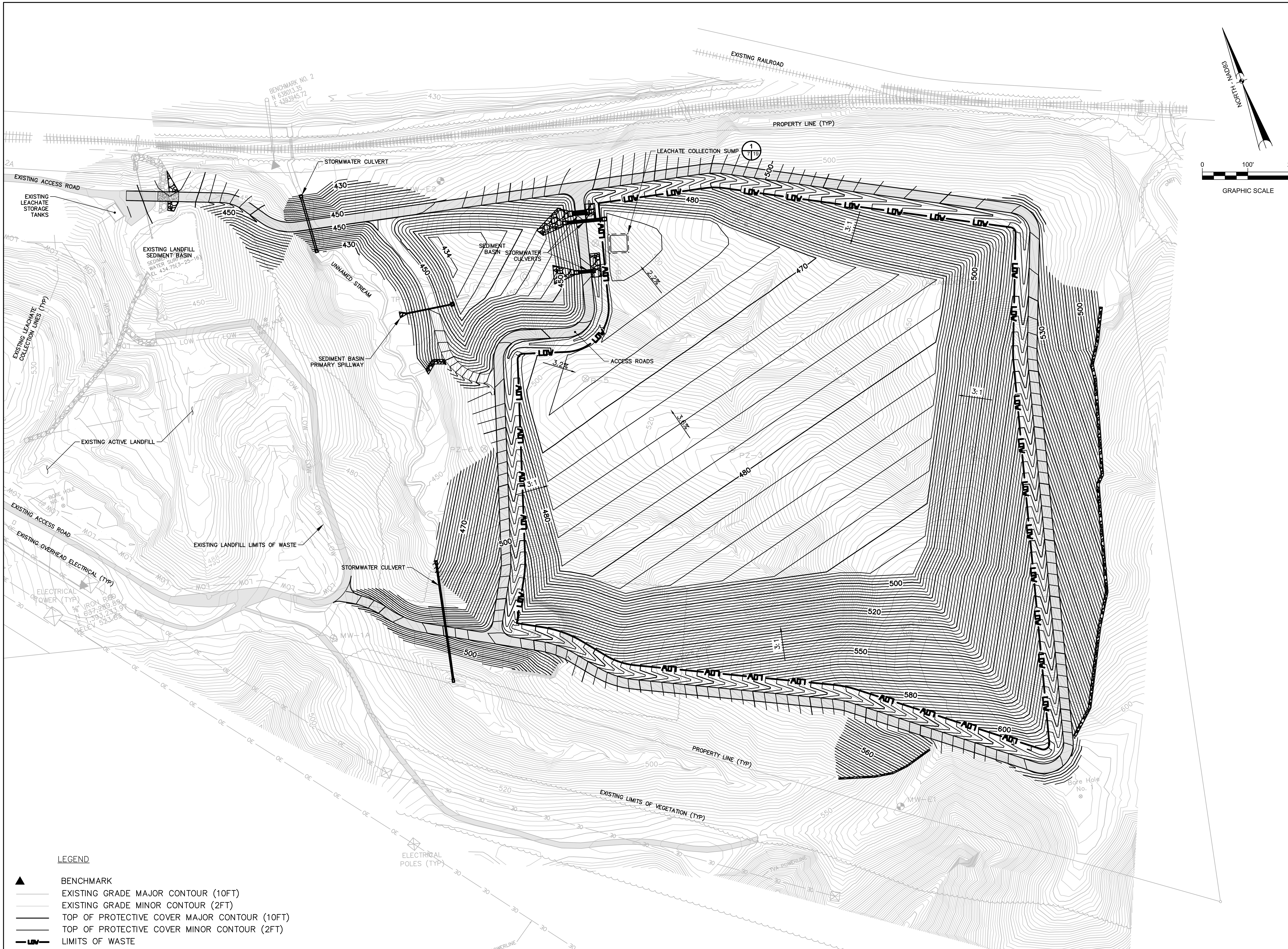

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REVISIONS

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EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

TOP OF PROTECTIVE COVER

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-7

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STATE OF TENNESSEE

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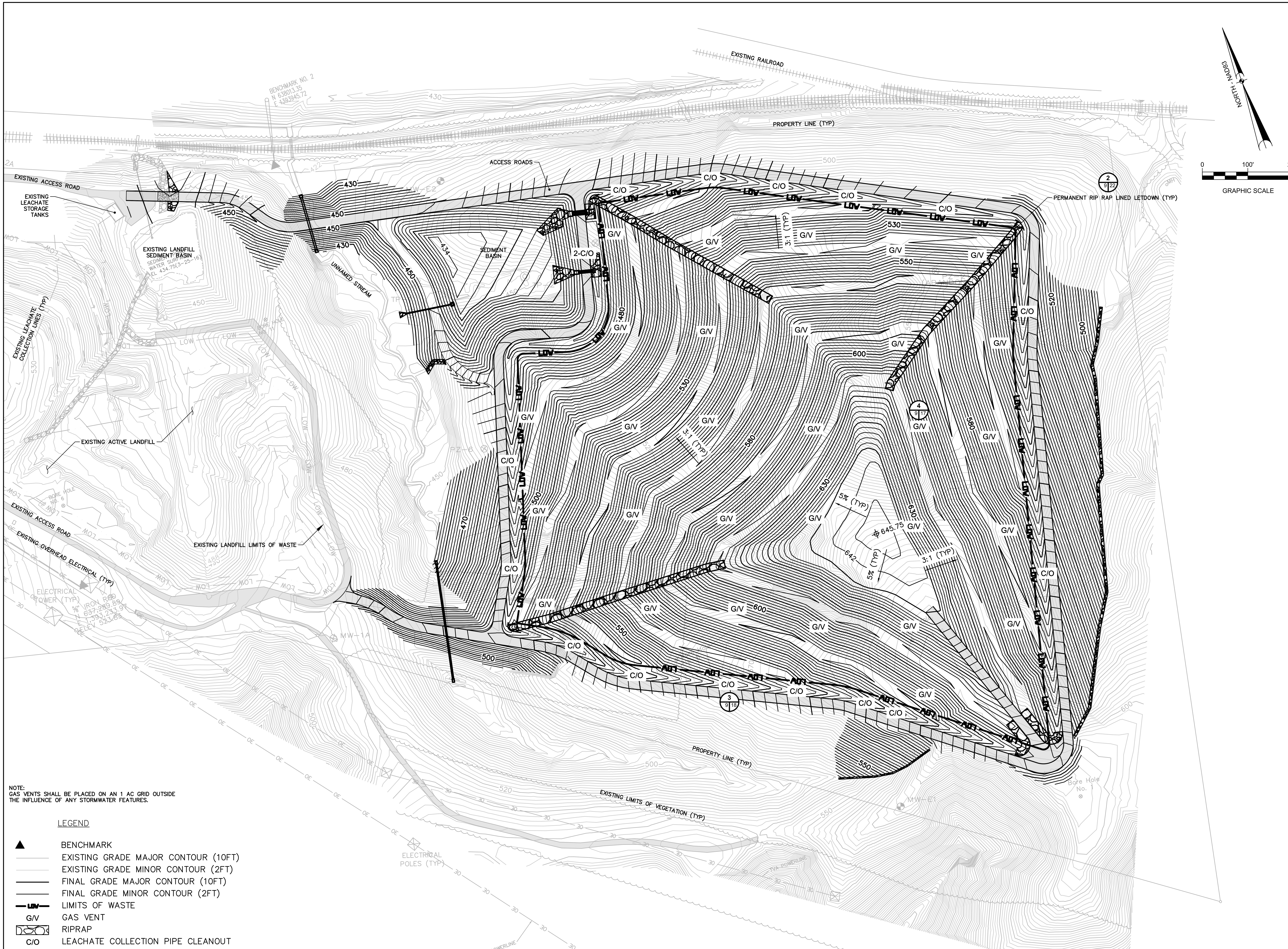
EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

TOP OF WASTE

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-8



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NO.	DATE	BY

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EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

FINAL GRADES

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
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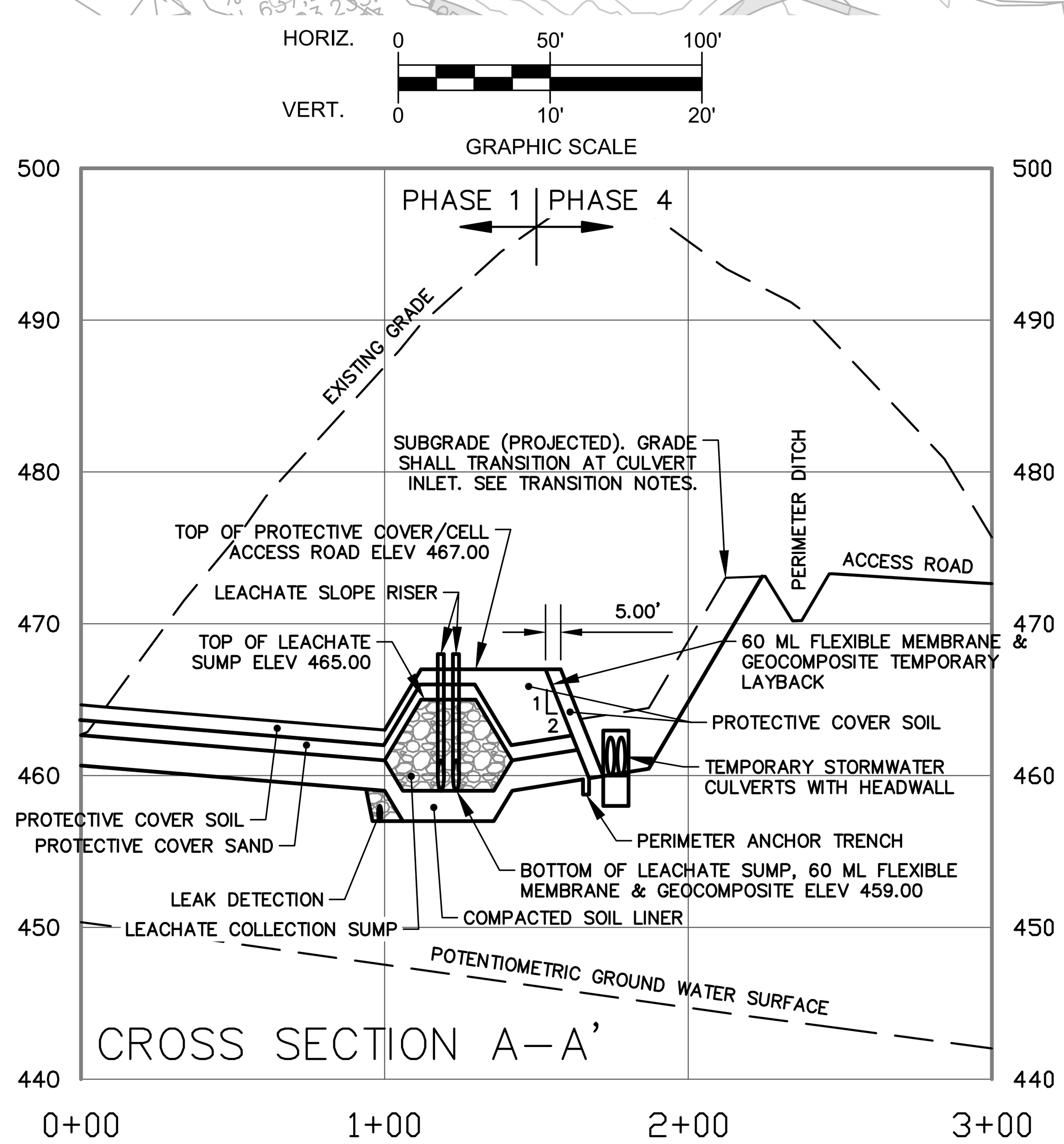
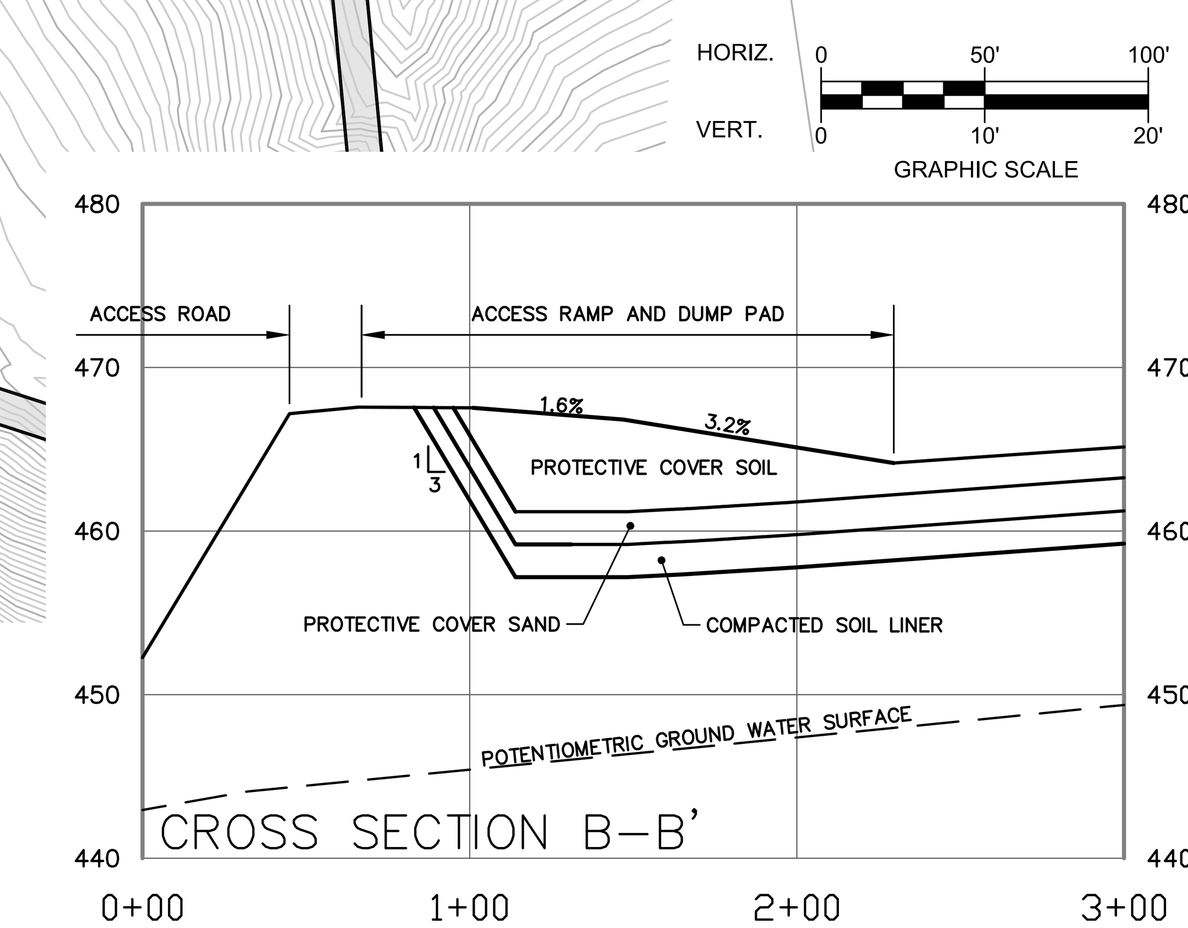
Franklin, TN 37067-6209

P-9





LEGEND

- BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (10FT)
- EXISTING GRADE MINOR CONTOUR (2FT)
- TOP OF PROTECTIVE COVER MAJOR CONTOUR (10FT)
- TOP OF PROTECTIVE COVER MINOR CONTOUR (2FT)
- LIMITS OF WASTE
- TACK-ON STORMWATER DIVERSION BERM
- CELL DIVERSION BERM
- STORMWATER FLOW DIRECTION
- FILL PROGRESSION
- RIPRAP

TRANSITION NOTE:
1. CULVERT INLET TRANSITION SHALL NOT EXCEED 2:1 SLOPE.
2. TRANSITION SURFACE SHALL BE 27" OF TDOT CLASS B RIP RAP.
3. NONWOVEN GEOTEXTILE FABRIC SHALL BE PLACED UNDER RIP RAP.

Culvert Table

Culvert ID	Peak Discharge (cfs)	Size/Material	Number Barrels - Length* per Barrel	Invert Elevation		Culvert Slope (%)*	Velocity (fps)	Riprap Apron Length (ft)
				Upstream	Downstream			
C3	42.34	30" RCP (Class V)	3 - 96	East 460.00	West 458.00	2.08	10.62	49

* Length and slope includes headwall and endwall where present

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NO.	DATE	BY

ISSUED FOR REVIEW DATE BY

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RECORD DRAWINGS DATE BY

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MICHAEL S. MAY
REGISTERED ENGINEER
AGRICULTURE COMMERCE
NO. 103104
STATE OF TENNESSEE

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EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

PHASE 1 DEVELOPMENT

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-10

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LEGEND

- ▲ BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (10FT)
- EXISTING GRADE MINOR CONTOUR (2FT)
- CELL 2 TOP OF PROTECTIVE COVER, CELL 1 TOP OF INTERMEDIATE COVER SOIL MAJOR CONTOUR (10FT)
- CELL 2 TOP OF PROTECTIVE COVER, CELL 1 TOP OF INTERMEDIATE COVER SOIL MINOR CONTOUR (2FT)
- LOW LIMITS OF WASTE
- TACK-ON STORMWATER DIVERSION BERM
- CELL DIVERSION BERM
- STORMWATER FLOW DIRECTION
- FILL PROGRESSION
- RIPRAP

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ADDENDUM NO.	ADDENDUM DATE	BY

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REVISIONS		
NO.	DATE	BY

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NO.	COMMENTS	DATE	BY
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EAST PHASE DISPOSAL FACILITY PERMIT DRAWINGS

PHASE 2 DEVELOPMENT

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-11

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LEGEND

▲	BENCHMARK
	EXISTING GRADE MAJOR CONTOUR (10FT)
	EXISTING GRADE MINOR CONTOUR (2FT)
	CELL 3 TOP OF PROTECTIVE COVER, CELL 1 AND 2 TOP OF INTERMEDIATE COVER SOIL MAJOR CONTOUR (10FT)
	CELL 3 TOP OF PROTECTIVE COVER, CELL 1 AND 2 TOP OF INTERMEDIATE COVER SOIL MINOR CONTOUR (2FT)
LOW	LIMITS OF WASTE
	TACK-ON STORMWATER DIVERSION BERM
	CELL DIVERSION BERM
	PARTIAL CLOSURE LIMITS
	STORMWATER FLOW DIRECTION
	FILL PROGRESSION
	RIPRAP

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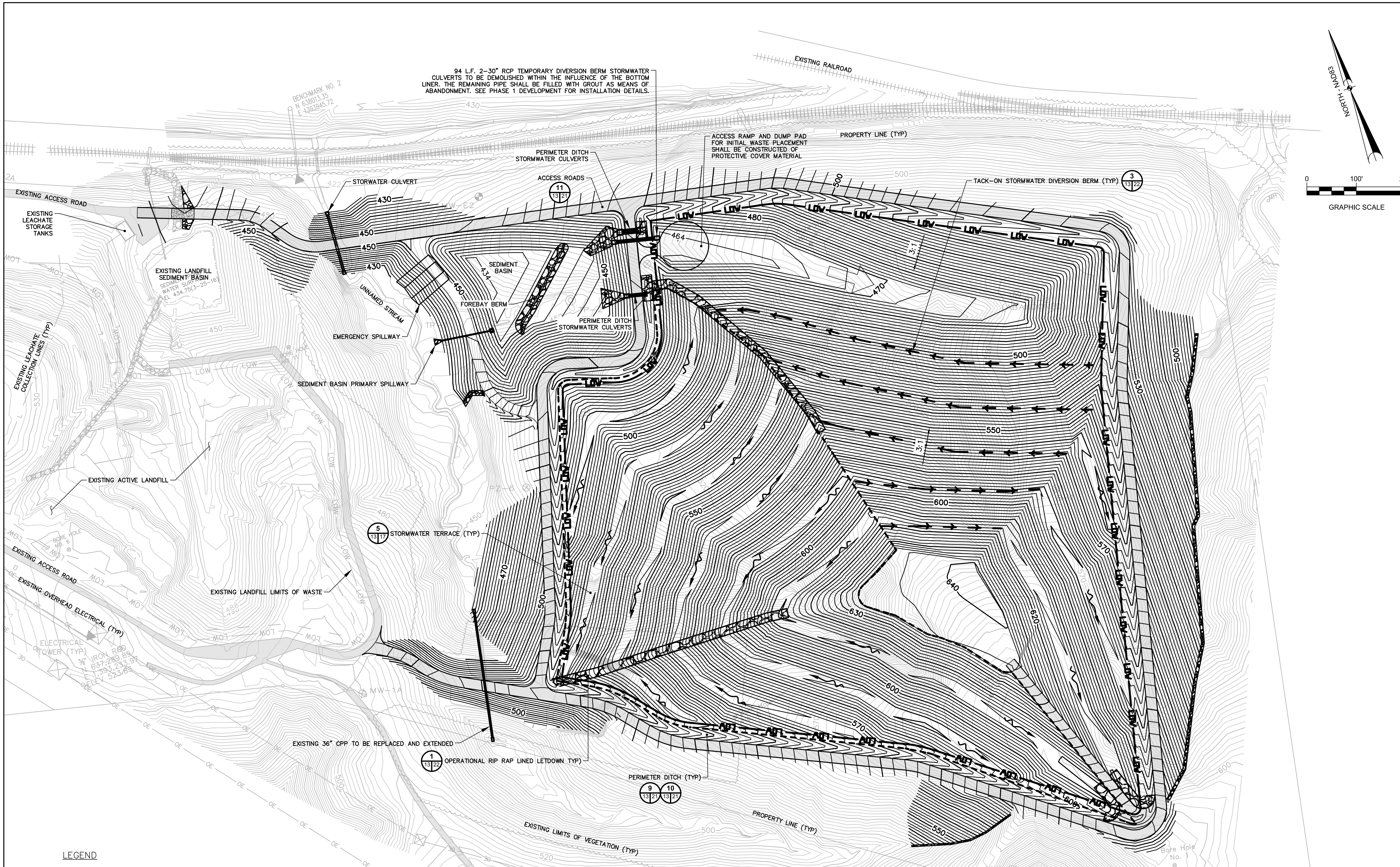
EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

PHASE 3 DEVELOPMENT

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-12



LEGEND

▲	BENCHMARK
	EXISTING GRADE MAJOR CONTOUR (10FT)
	EXISTING GRADE MINOR CONTOUR (2FT)
	CELL 4 TOP OF PROTECTIVE COVER, CELL 1, 2 AND 3 TOP OF INTERMEDIATE COVER SOIL MAJOR CONTOUR (10FT)
	CELL 4 TOP OF PROTECTIVE COVER, CELL 1, 2 AND 3 TOP OF INTERMEDIATE COVER SOIL MINOR CONTOUR (2FT)
—LOW—	LIMITS OF WASTE
	TACK-ON STORMWATER DIVERSION BERM
	CELL DIVERSION BERM
	PARTIAL CLOSURE LIMITS
	STORMWATER FLOW DIRECTION
	FILL PROGRESSION
	RIPRAP

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EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

PHASE 4 DEVELOPMENT

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

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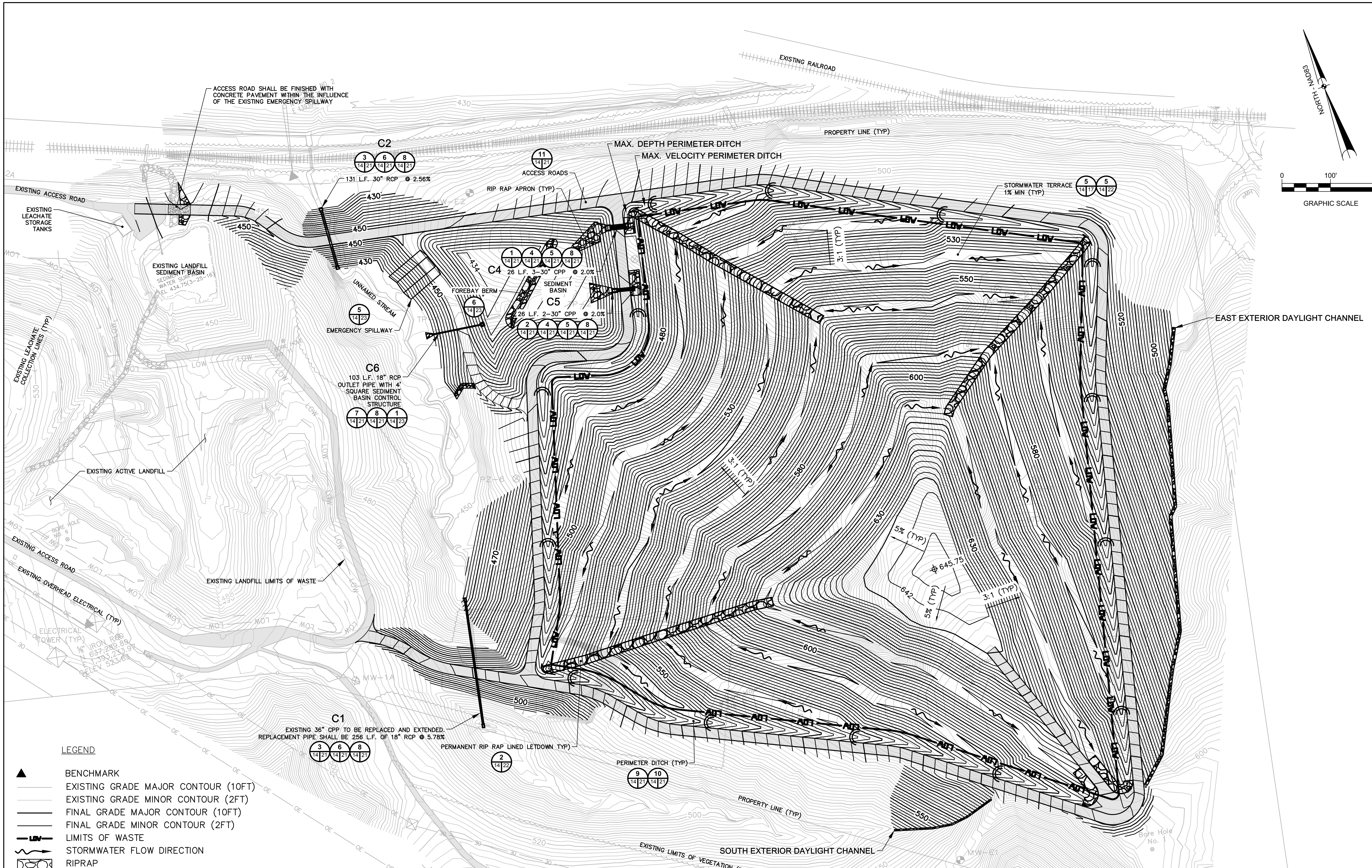

LEGEND

- BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (10FT)
- EXISTING GRADE MINOR CONTOUR (2FT)
- FINAL GRADE MAJOR CONTOUR (10FT)
- FINAL GRADE MINOR CONTOUR (2FT)
- LIMITS OF WASTE
- STORMWATER FLOW DIRECTION
- RIPRAP
- ROCK CHECK DAM

Culvert Table

Culvert ID	Peak Discharge (cfs)	Size/Material	Number Barrels - Length* per Barrel	Invert Elevation Upstream	Invert Elevation Downstream	Culvert Slope (%)*	Velocity (fps)	Riprap Apron Length (ft)
C1	21.9	18" RCP (Class V)	1 - 270	South 468.01	North 455.00	4.82	14.84	N/A
C2	36.87	30" RCP (Class V)	1 - 131	South 424.82	North 422.27	1.95	12.15	N/A
C3	See Sheet P-10							
C4	83.65	30" CPP	3 - 46	East 463.75	West 462.75	2.17	11.41	37
C5	58.35	30" CPP	2 - 46	East 461.25	West 460.25	2.17	11.51	46
C6	3.97	18" RCP (Class V)	1 - 105	East 438.75	West 434.38	4.16	9.48	13

* Length and slope includes headwall and endwall where present

Channel Table

Parameter	Q_{25} Peak Discharge (cfs)	Slope (ft/ft)	Flow Depth (feet)	Freeboard (feet)	Velocity (fps)	Shear Stress (lb/ft^2)	Liner
Max. Depth Perimeter Ditch	83.65	0.02	1.709	1.291	6.025	1.255	TRM
Max. Velocity Perimeter Ditch	83.65	0.08	1.237	1.763	10.071	3.83	TRM
South Exterior Daylight	3.6	0.26	0.383	N/A	7.006	2.987	A-1 Riprap
East Exterior Daylight	25.26	0.12	0.978	N/A	8.811	3.472	A-1 Riprap

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EAST PHASE DISPOSAL FACILITY PERMIT DRAWINGS

STORMWATER MANAGEMENT PLAN

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-14

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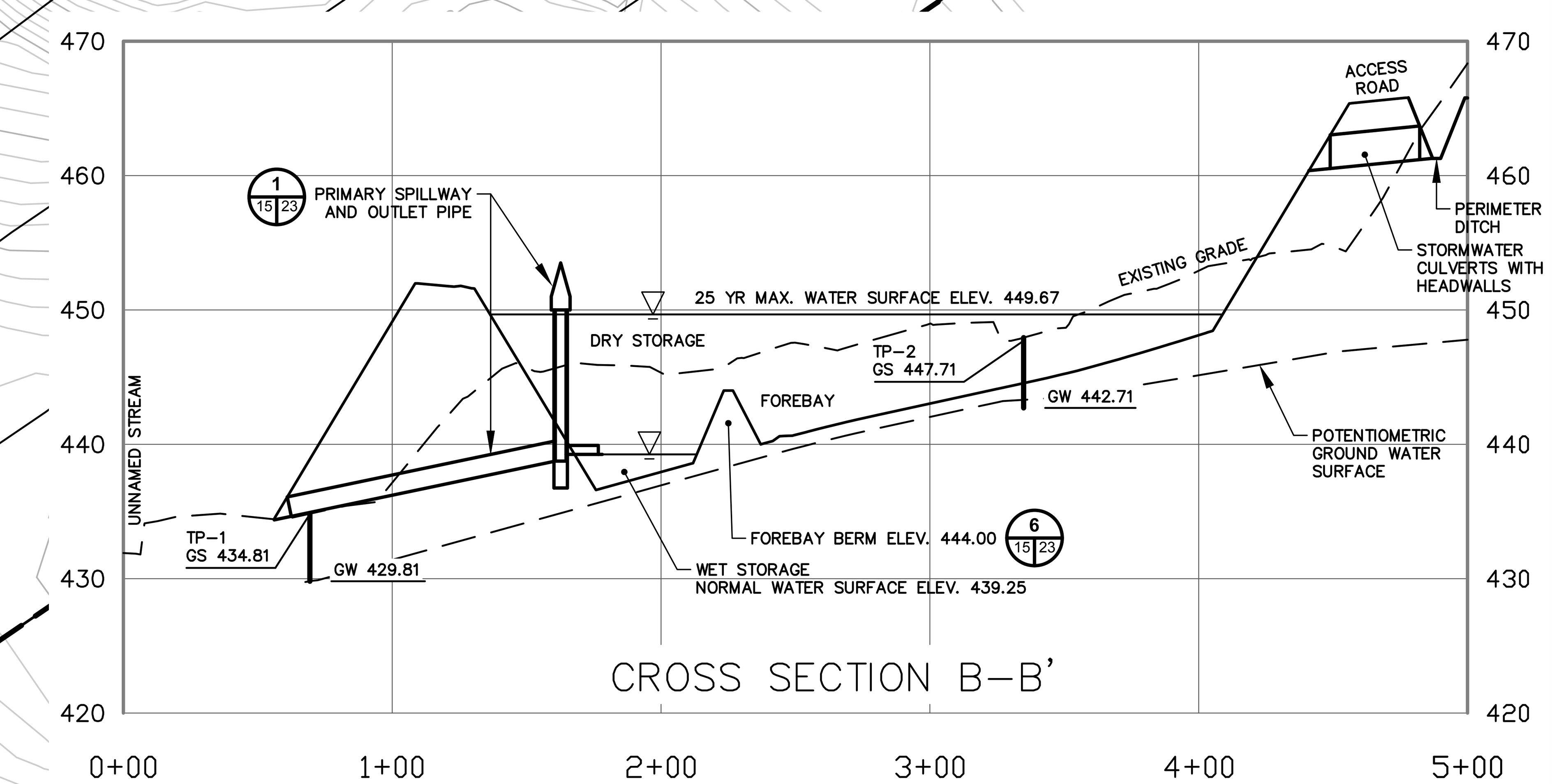
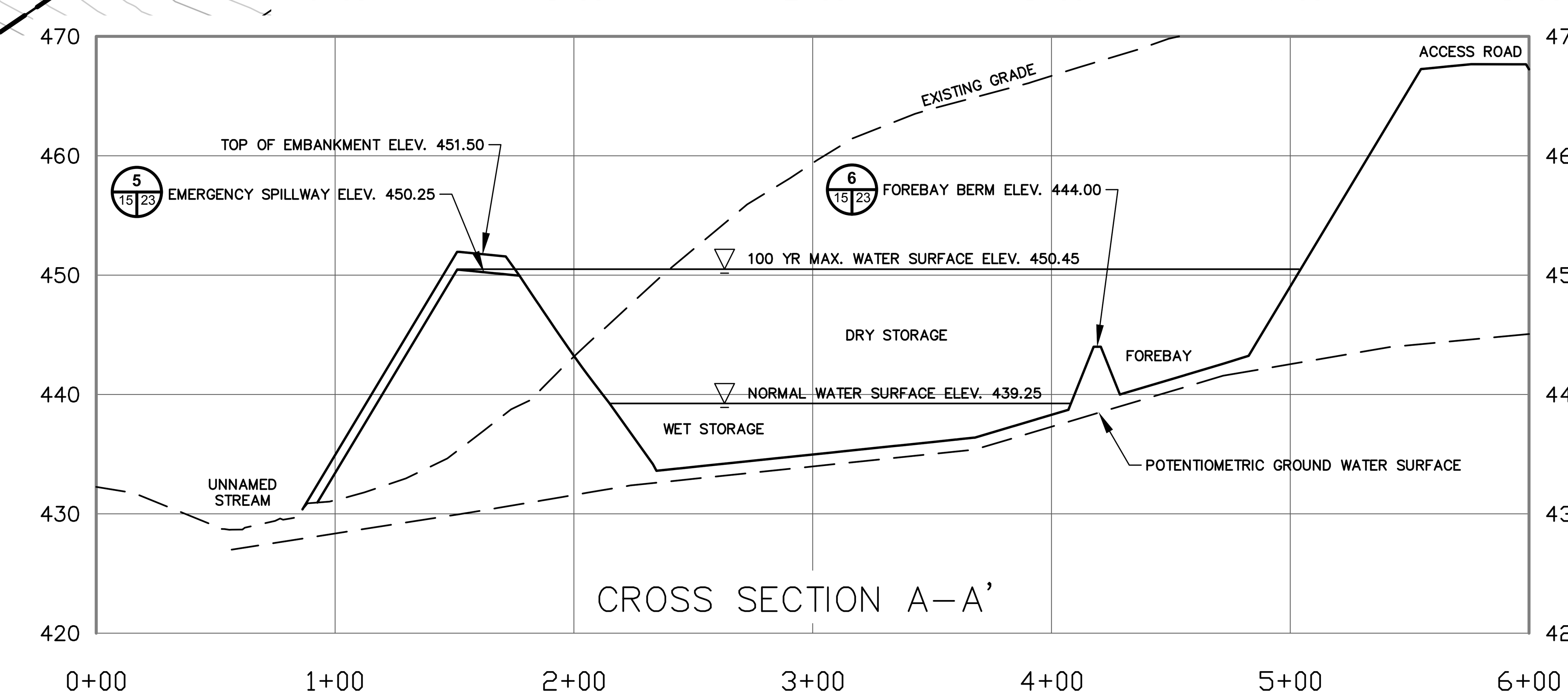
HORIZ. 0 50' 100'
VERT. 0 10' 20'
GRAPHIC SCALE

LEGEND

- 1989 BOREHOLE
- B-1 2016 BOREHOLE
- PZ-3 2016 PIEZOMETER
- MW-E1 2016 BOREHOLE/M.W.
- 2016 GROUND WATER TEST PIT
- EXISTING M.W.
- BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (10FT)
- EXISTING GRADE MINOR CONTOUR (2FT)
- TOP OF PROPOSED GRADE MAJOR CONTOUR (10FT)
- TOP OF PROPOSED GRADE MINOR CONTOUR (2FT)
- GROUNDWATER POTENTIOMETRIC CONTOUR
- LOW LIMITS OF WASTE
- RIPRAP



CROSS SECTION B-B'



CROSS SECTION A-A'

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EAST PHASE DISPOSAL FACILITY PERMIT DRAWINGS

SEDIMENT BASIN

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-15

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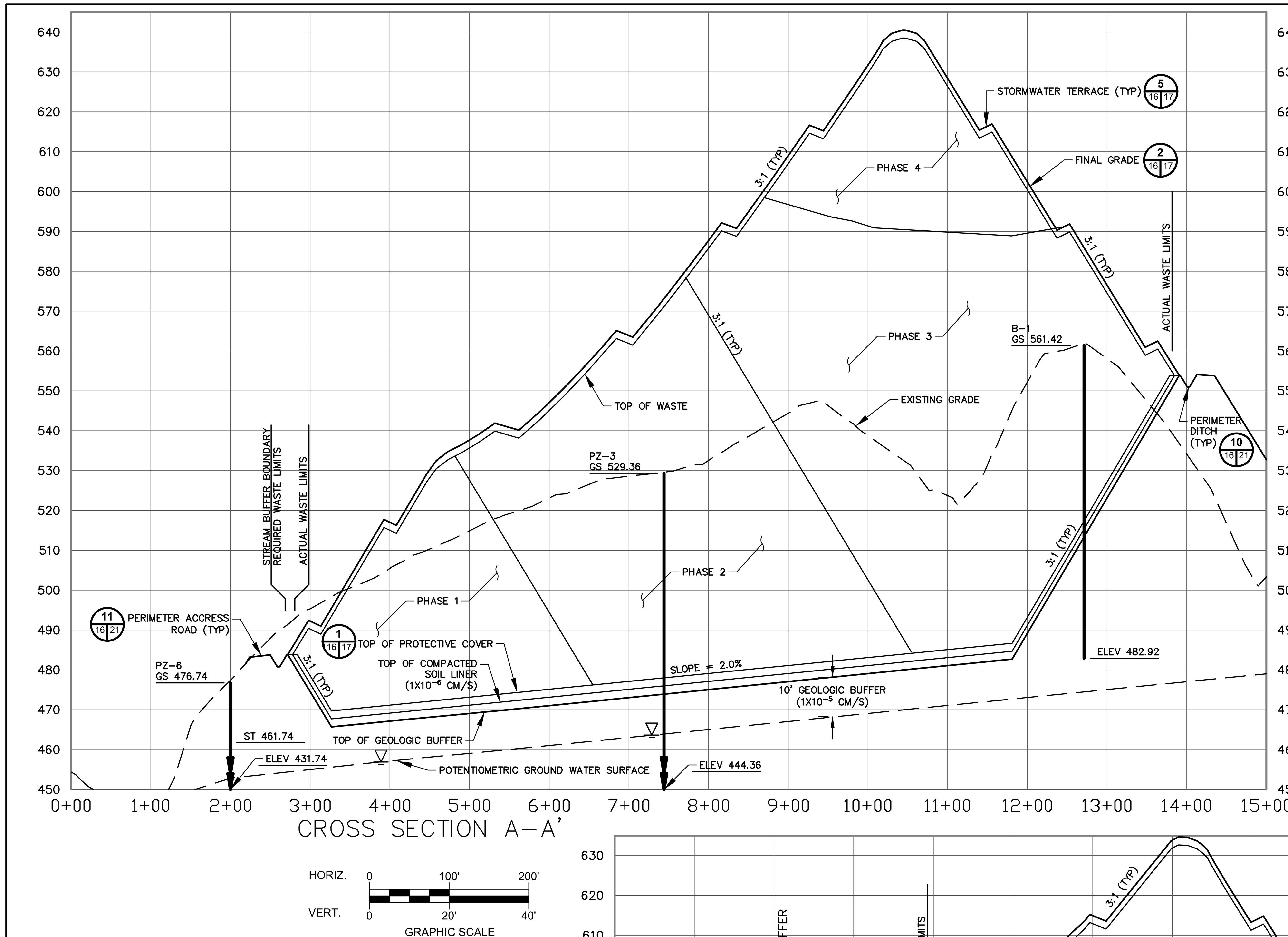

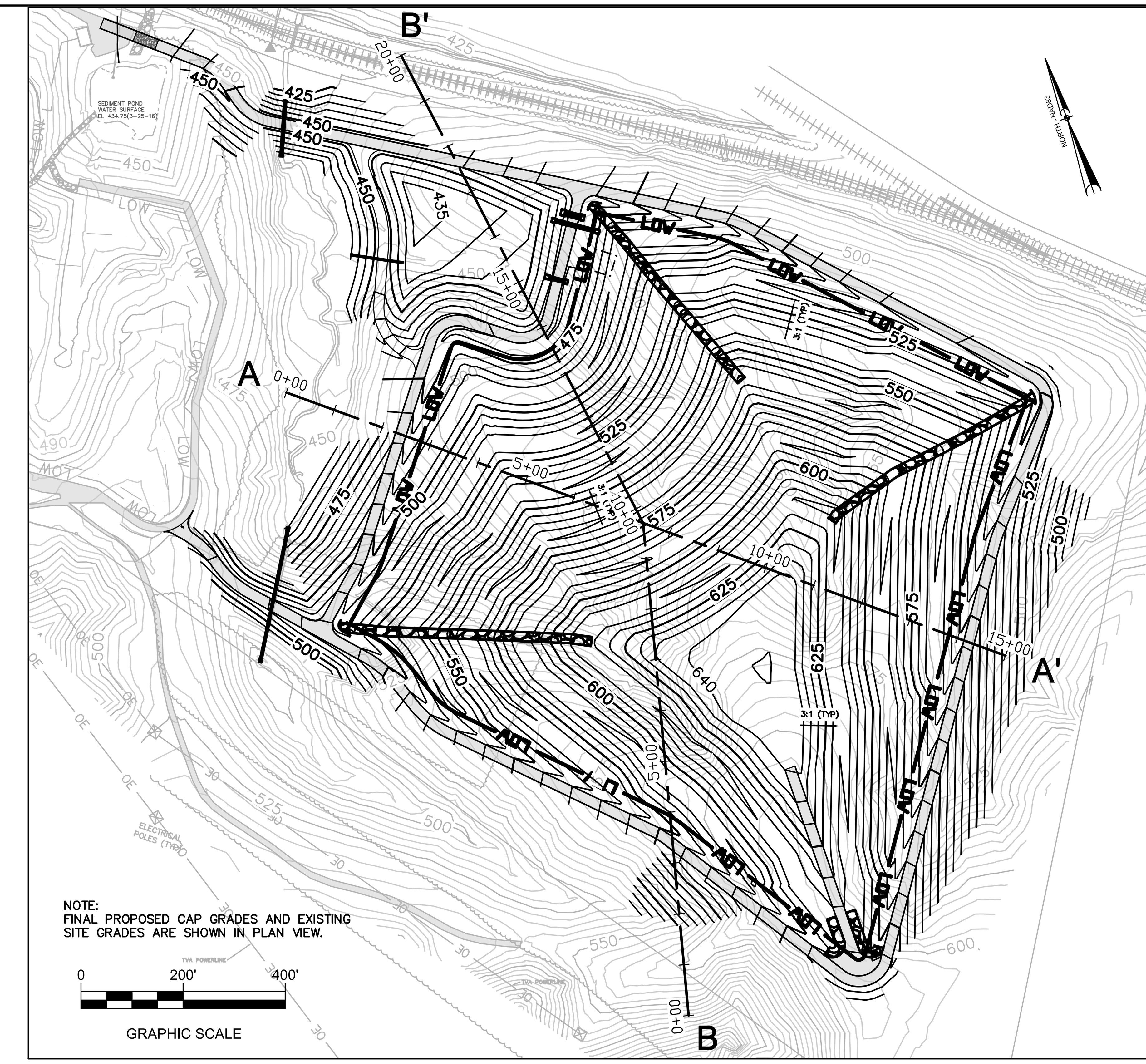

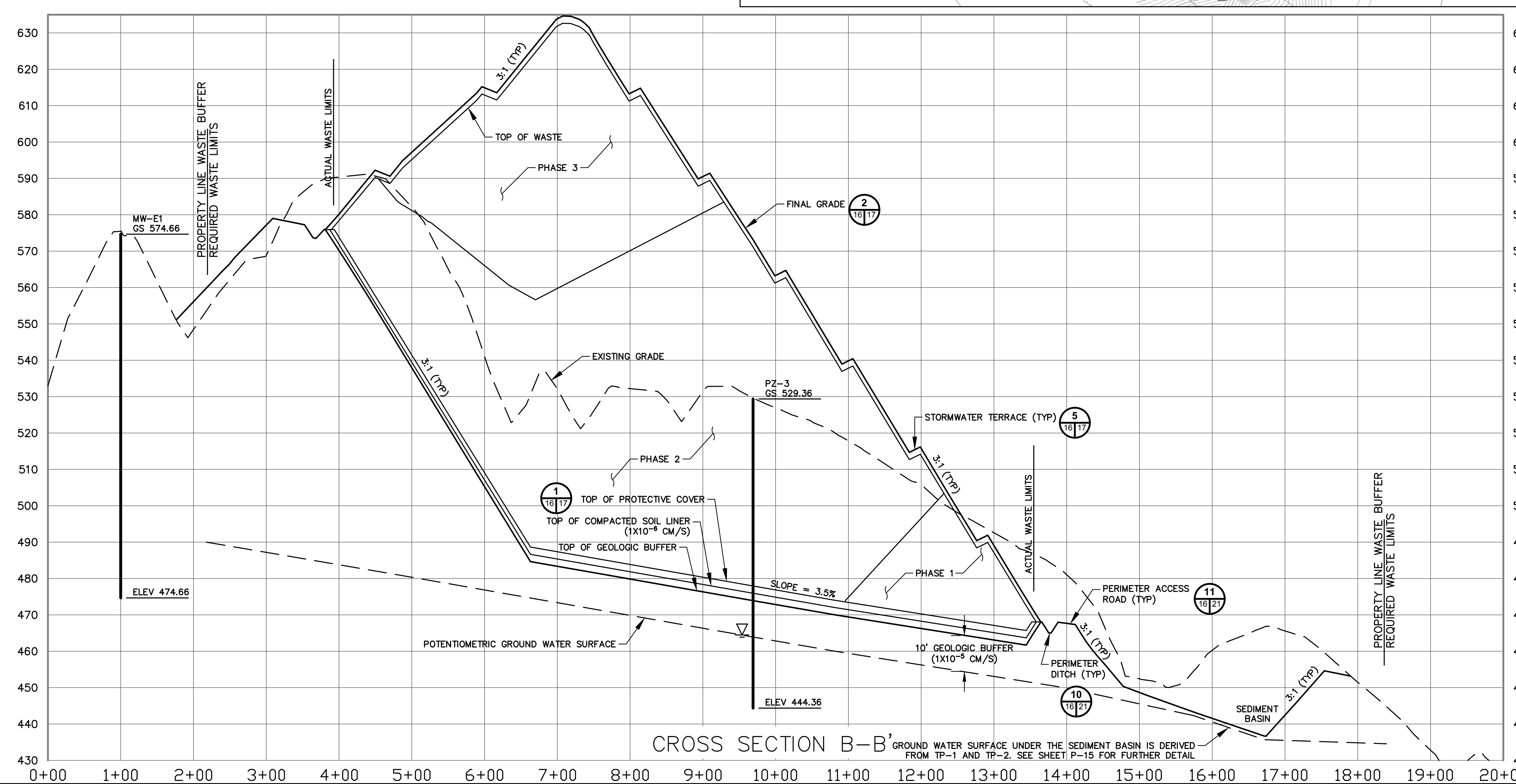

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ADDENDUM REVISIONS

ADDENDUM NO.	ADDENDUM DATE	BY

ISSUED FOR CONSTRUCTION ___ DATE BY

REVISIONS

NO.	DATE	BY

ISSUED FOR REVIEW ___ DATE BY

REVISIONS

NO.	COMMENTS	DATE	BY
0	ISSUED FOR PERMITTING	7/1/16	NSP
1	LEACHATE TANK, ROAD SECTION	8/10/16	NSP
2	TDEC PERMIT REVIEW COMMENTS	1/10/17	NSP
3	TDEC PERMIT REVIEW COMMENTS	5/9/17	NSP
4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

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SCALE AS SHOWN	

EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

CROSS SECTIONS

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-16

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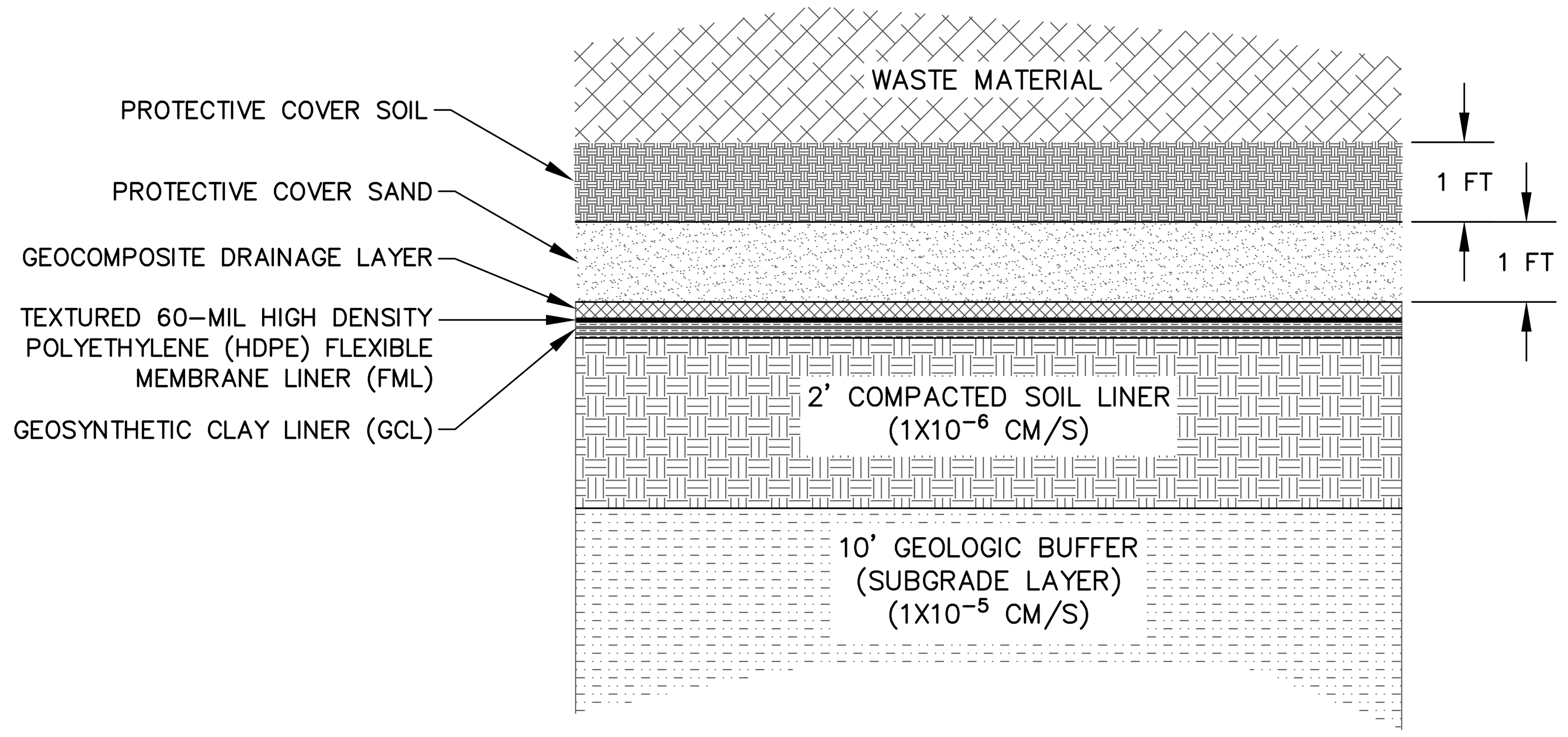
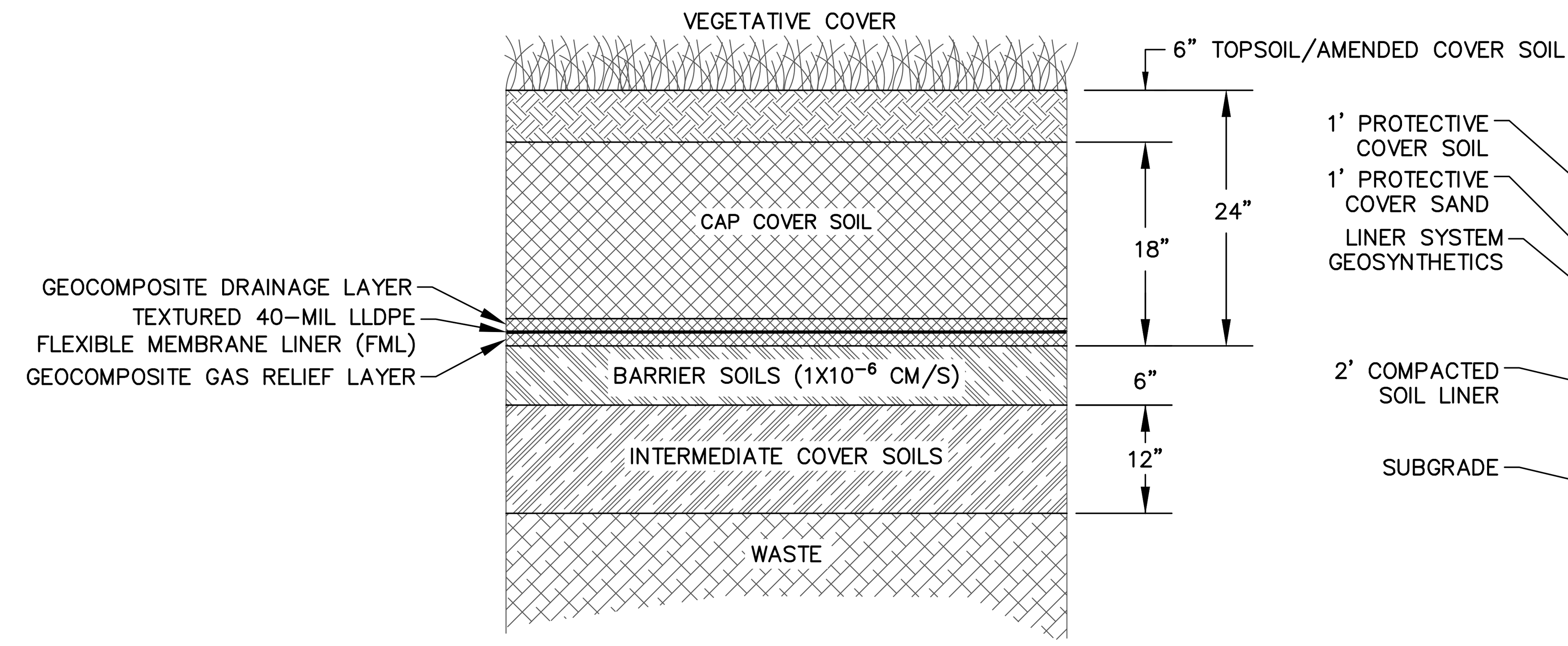


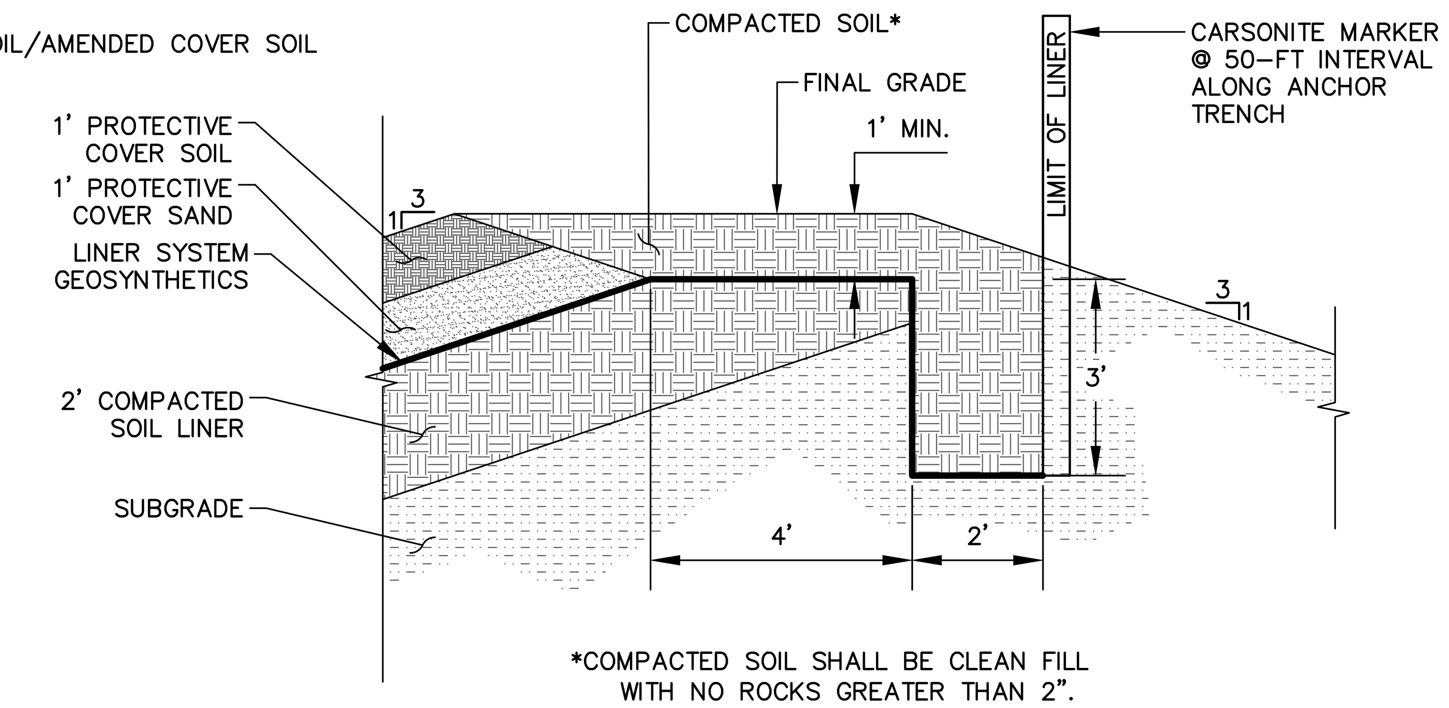
1 LINER SYSTEM
16 17 / 17 / 18 / 19
NOT TO SCALE

NOTE:
1. LINER SYSTEM TO BE TEXTURED 60-MIL HIGH DENSITY POLYETHYLENE (HDPE).
2. COMPACTED SOIL LAYER TO BE CLASSIFIED AS CL, CH, SC, SM, CL-ML, OR GC AND COMPACTED TO 95% OF MAXIMUM DRY DENSITY AS DETERMINED BY ASTM D698.
3. UPPER 4" OF COMPACTED SOIL LAYER TO BE SCREENED, HAND PICKED, OR MECHANICALLY PICKED TO REMOVE ROCKS GREATER THAN 2".
4. PROTECTIVE COVER SOIL TO BE NON-COHESIVE SOIL OR COHESIVE SITE SOILS. IF COHESIVE SITE SOILS ARE TO BE UTILIZED, TRENCHES BACKFILLED WITH GRANULAR MATERIAL ARE TO BE CONSTRUCTED ABOVE AND ALONG THE LEACHATE COLLECTION PIPE.
5. GEOCOMPOSITE DRAINAGE LAYER SHALL BE PROTECTED FROM PHOTODEGRADATION AND FREEZE/THAW EFFECTS IN ACCORDANCE WITH MANUFACTURE'S RECOMMENDATIONS.



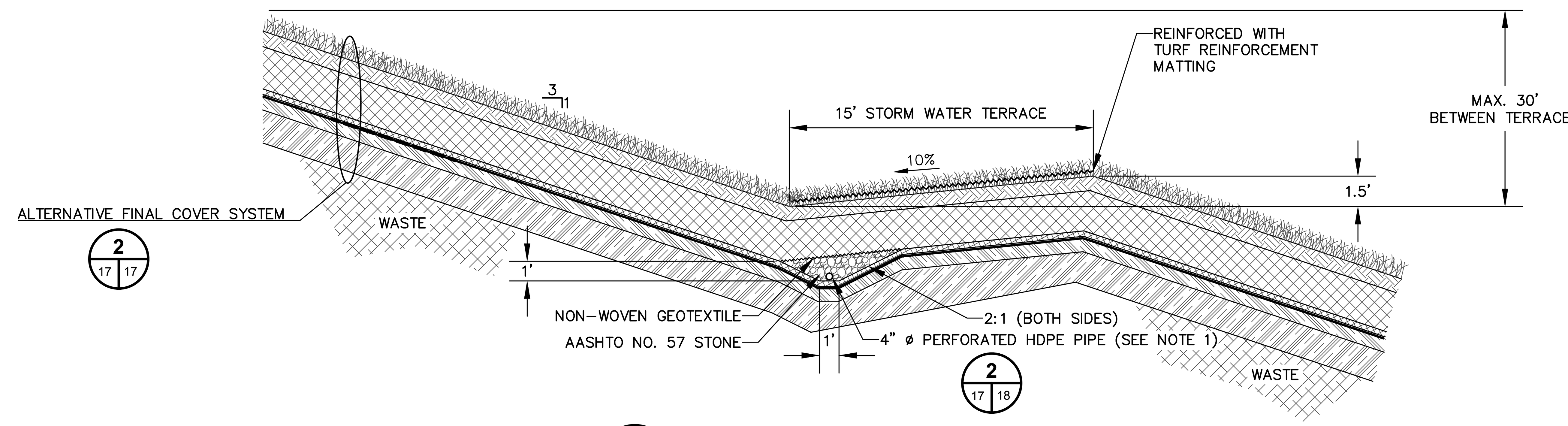
2 ALTERNATIVE FINAL COVER SYSTEM
16 17 / 17 / 18 / 22
NOT TO SCALE

NOTE:
1. GEOCOMPOSITE DRAINAGE LAYER SHALL BE PROTECTED FROM PHOTODEGRADATION AND FREEZE/THAW EFFECTS IN ACCORDANCE WITH MANUFACTURE'S RECOMMENDATIONS.



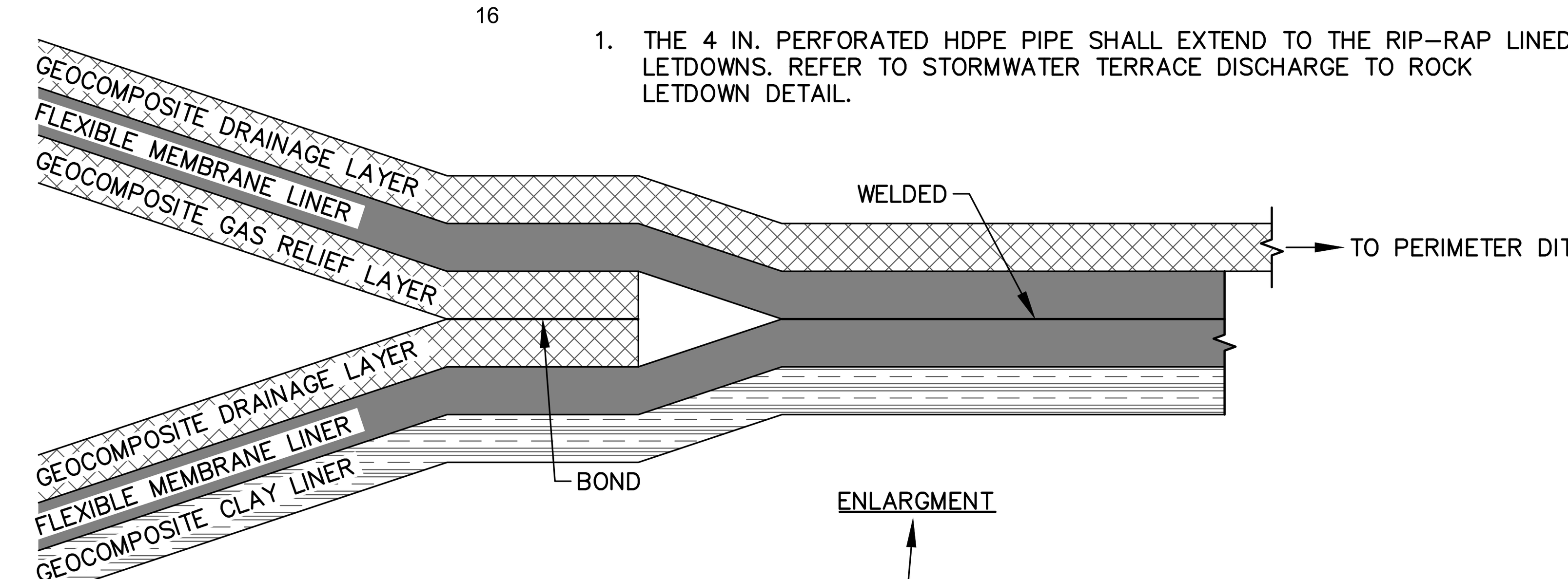
*COMPACTED SOIL SHALL BE CLEAN FILL WITH NO ROCKS GREATER THAN 2".

3 PERIMETER ANCHOR TRENCH
17 17
NOT TO SCALE

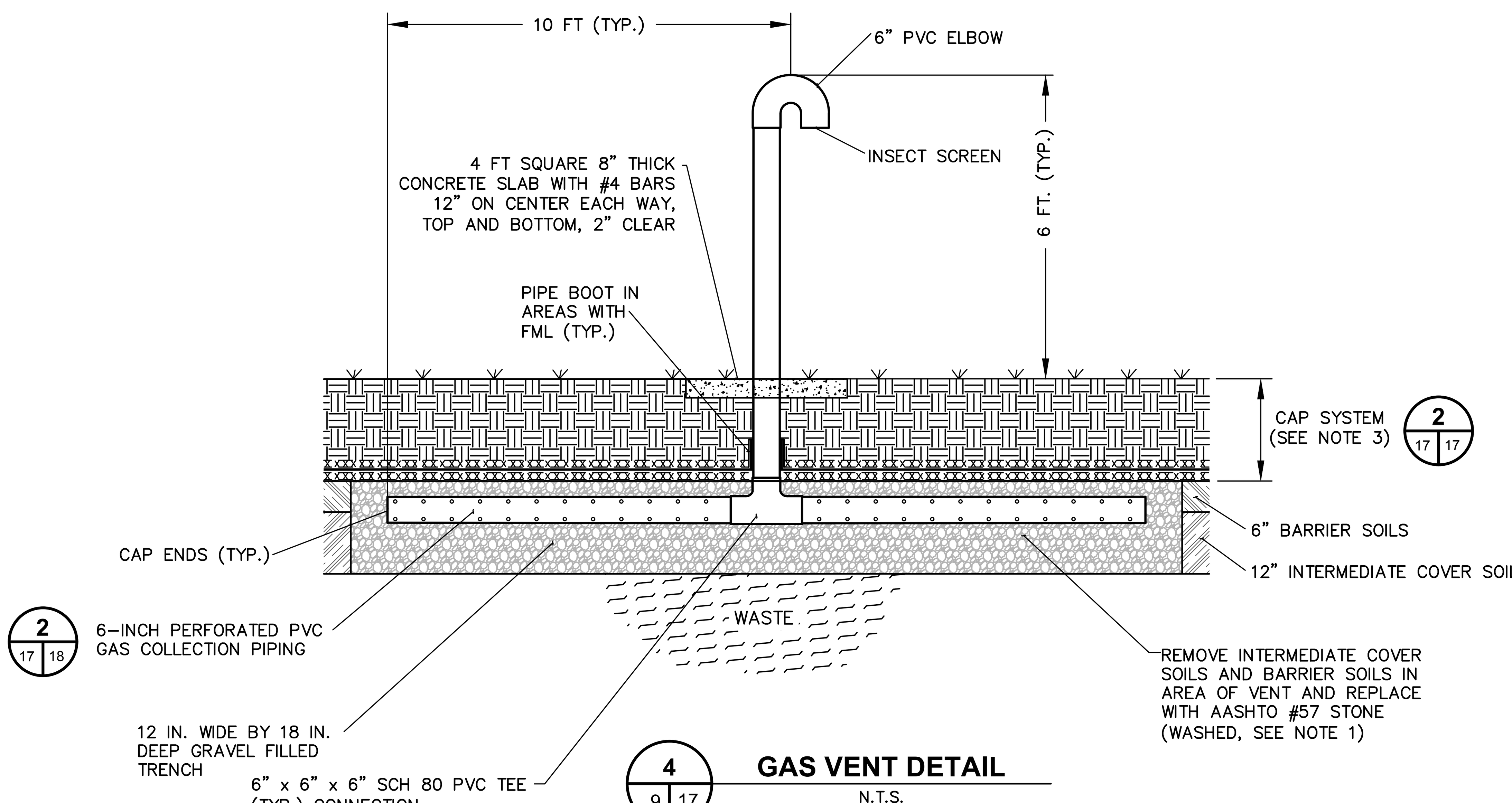


5 STORM WATER TERRACE
11 17 / 12 / 13 / 14 / 16
NOT TO SCALE

NOTES:
1. THE 4 IN. PERFORATED HDPE PIPE SHALL EXTEND TO THE RIP-RAP LINED LETDOWNS. REFER TO STORMWATER TERRACE DISCHARGE TO ROCK LETDOWN DETAIL. 5 / 17 22

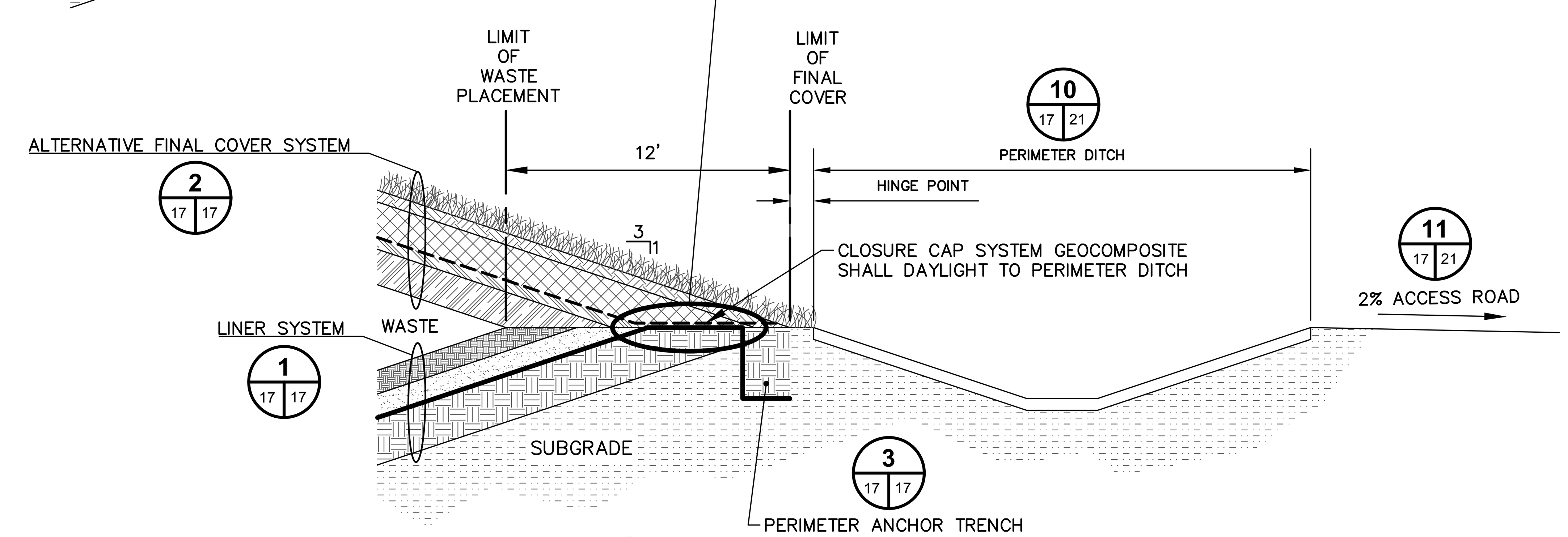


ENLARGMENT



4 GAS VENT DETAIL
9 17
N.T.S.

NOTES
1. THE AASHTO #57 STONE USED IN THE LANDFILL GAS WELLS AND LATERAL TRENCHES SHALL BE WASHED PER AASHTO T11 AND CONTAIN LESS THAN 1.50% MATERIALS FINER THAN THE NO.200 SIEVE.
2. GAS VENTS SHOULD BE ORIENTED WITH GAS PIPING PERPENDICULAR TO SLOPE.
3. GAS VENTS SHALL BE PLACED IN THE APPROXIMATE LOCATIONS AS SHOWN ON SHEET P-9.



6 PERIMETER BERM AND CAP TIE-IN
17 17
NOT TO SCALE

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ISSUED FOR REVIEW DATE BY

REVISIONS

NO.	COMMENTS	DATE	BY
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5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

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EAST PHASE DISPOSAL FACILITY
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LINER, CAP, AND COVER SYSTEM DETAILS

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
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P-17

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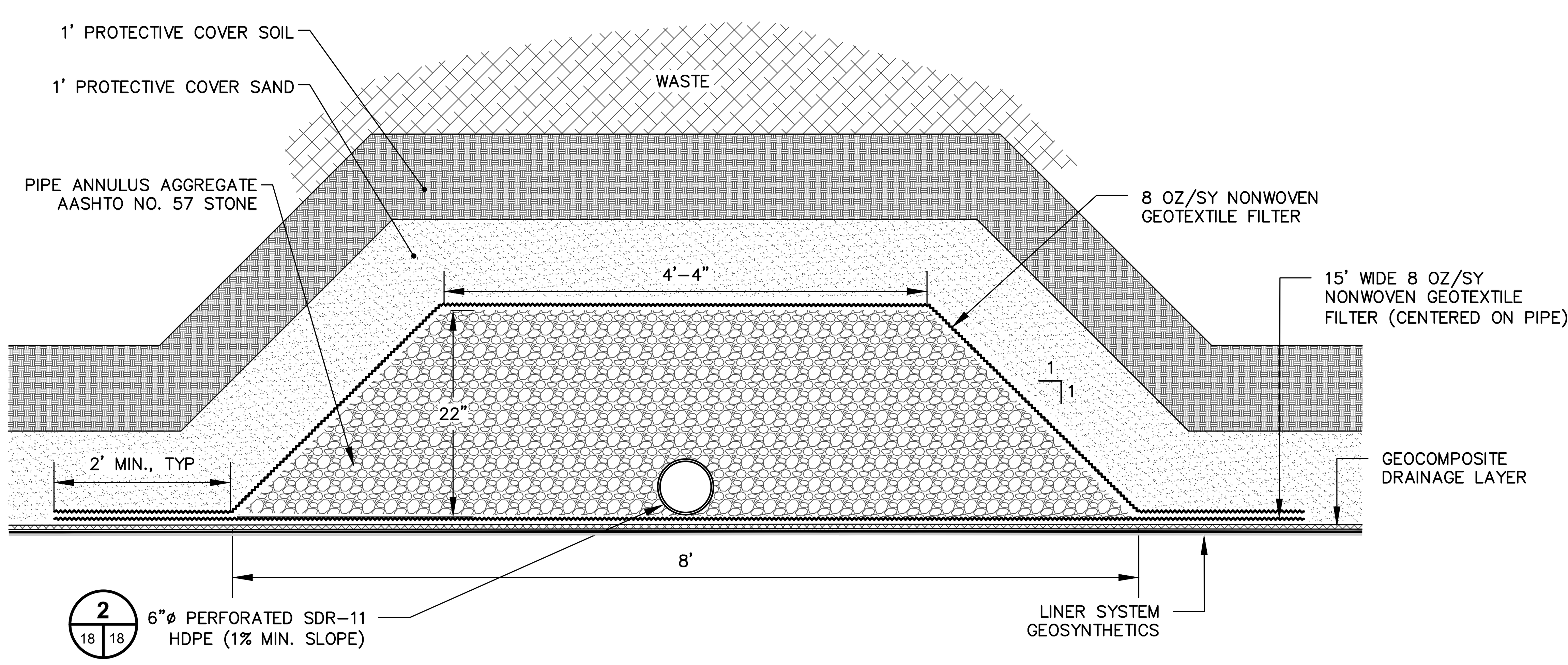
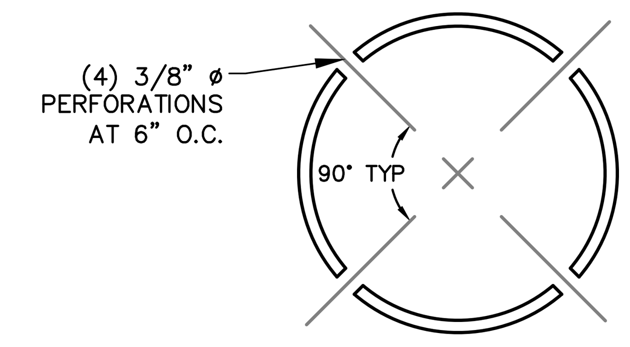
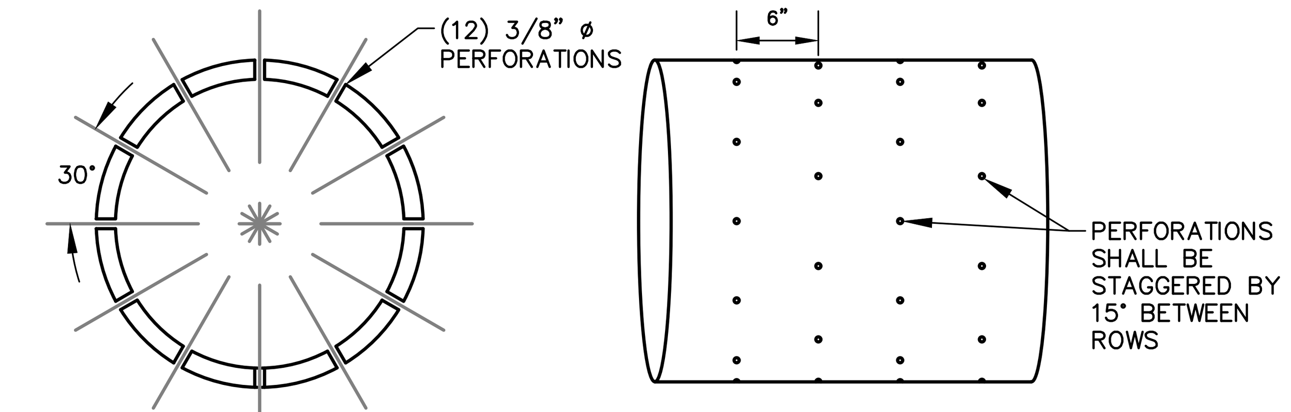


1 6 18 18 19 22 **LEACHATE COLLECTION PIPE**
NOT TO SCALE

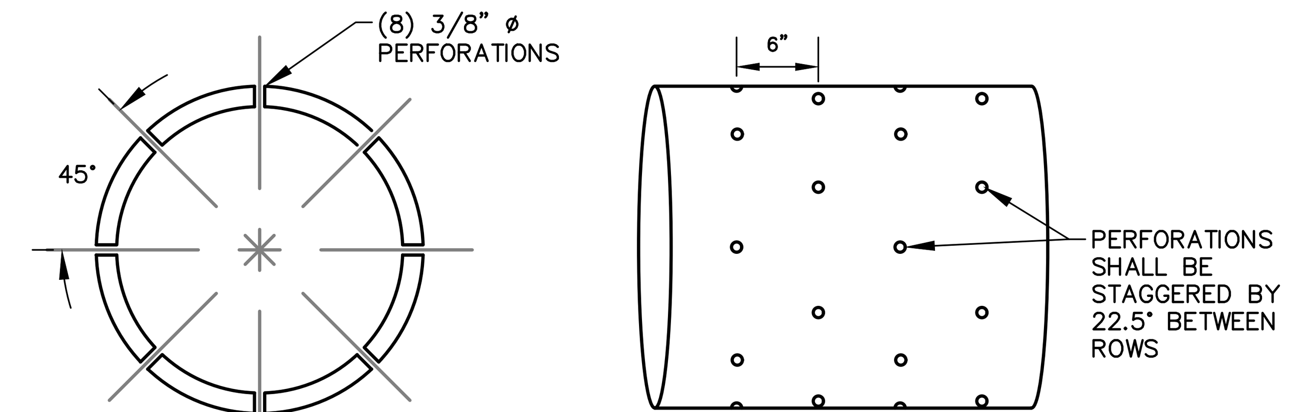
LEACHATE AGGREGATE CARBONATE CONTENT NOTE:
MAXIMUM CARBONATE CONTENT OF THE LEACHATE COLLECTION SYSTEM AGGREGATE SHALL BE 15%. CARBON CONTENT SHALL BE DETERMINED IN ACCORDANCE WITH ASTM-3042. ASTM-3042 SHALL BE CONDUCTED WITH NO pH MODIFICATION SOLUTION (6N HCL).



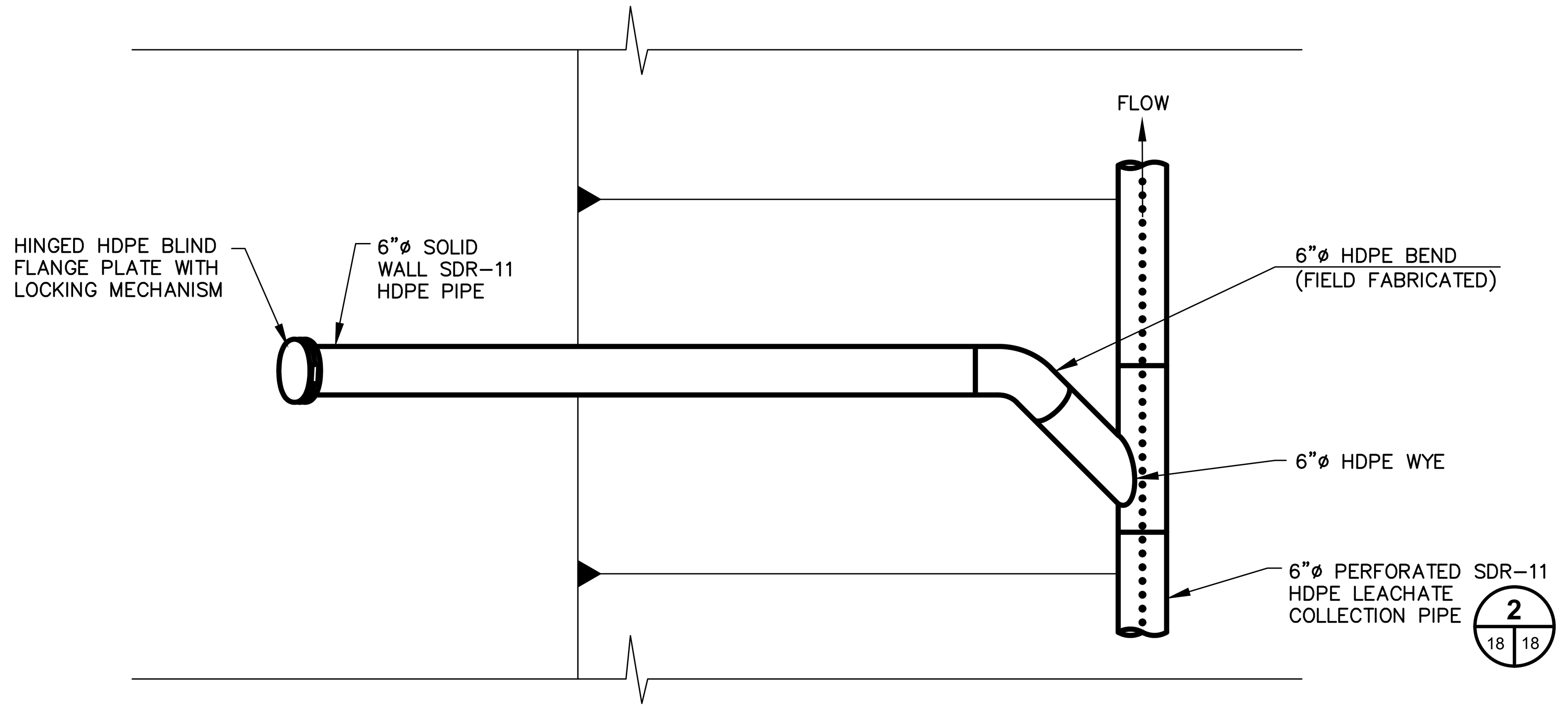
2 6 18 17 18 19 22 **HDPE PERFORATION**
NOT TO SCALE



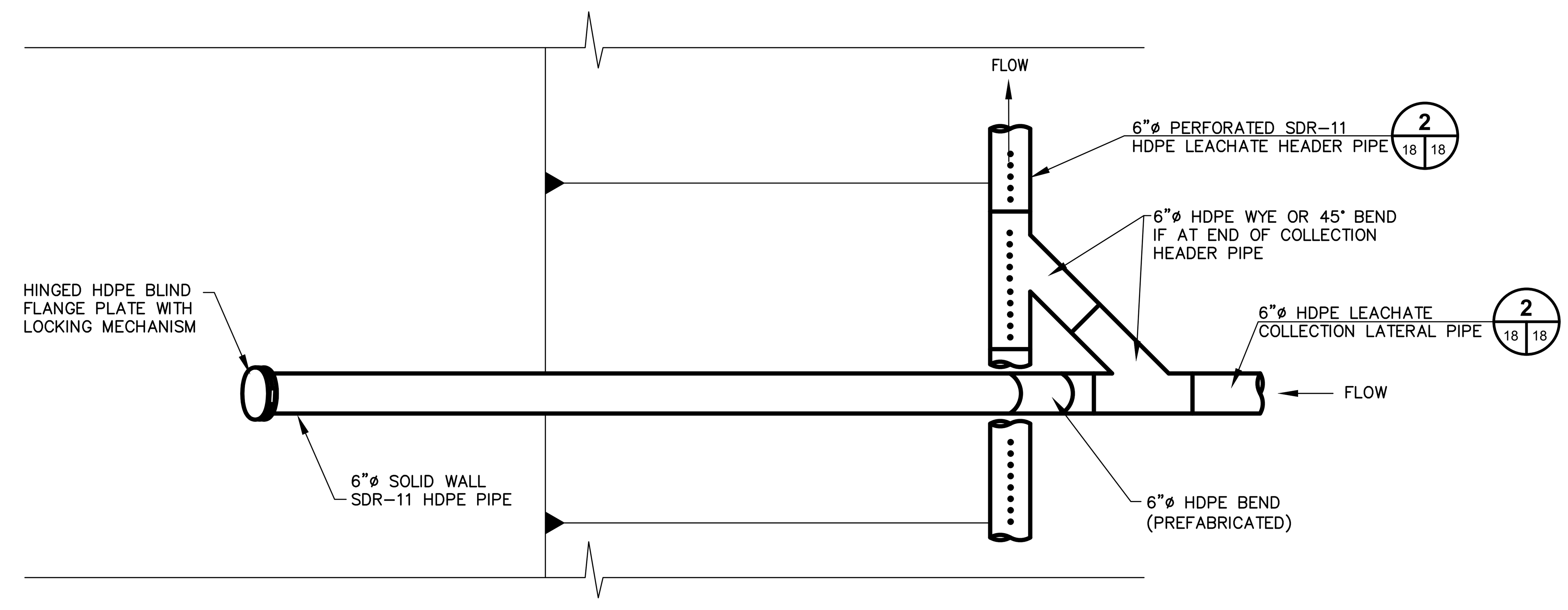
4 19 18 **24" HDPE PERFORATION**
NOT TO SCALE



5 19 18 **12" HDPE PERFORATION**
NOT TO SCALE



PLAN



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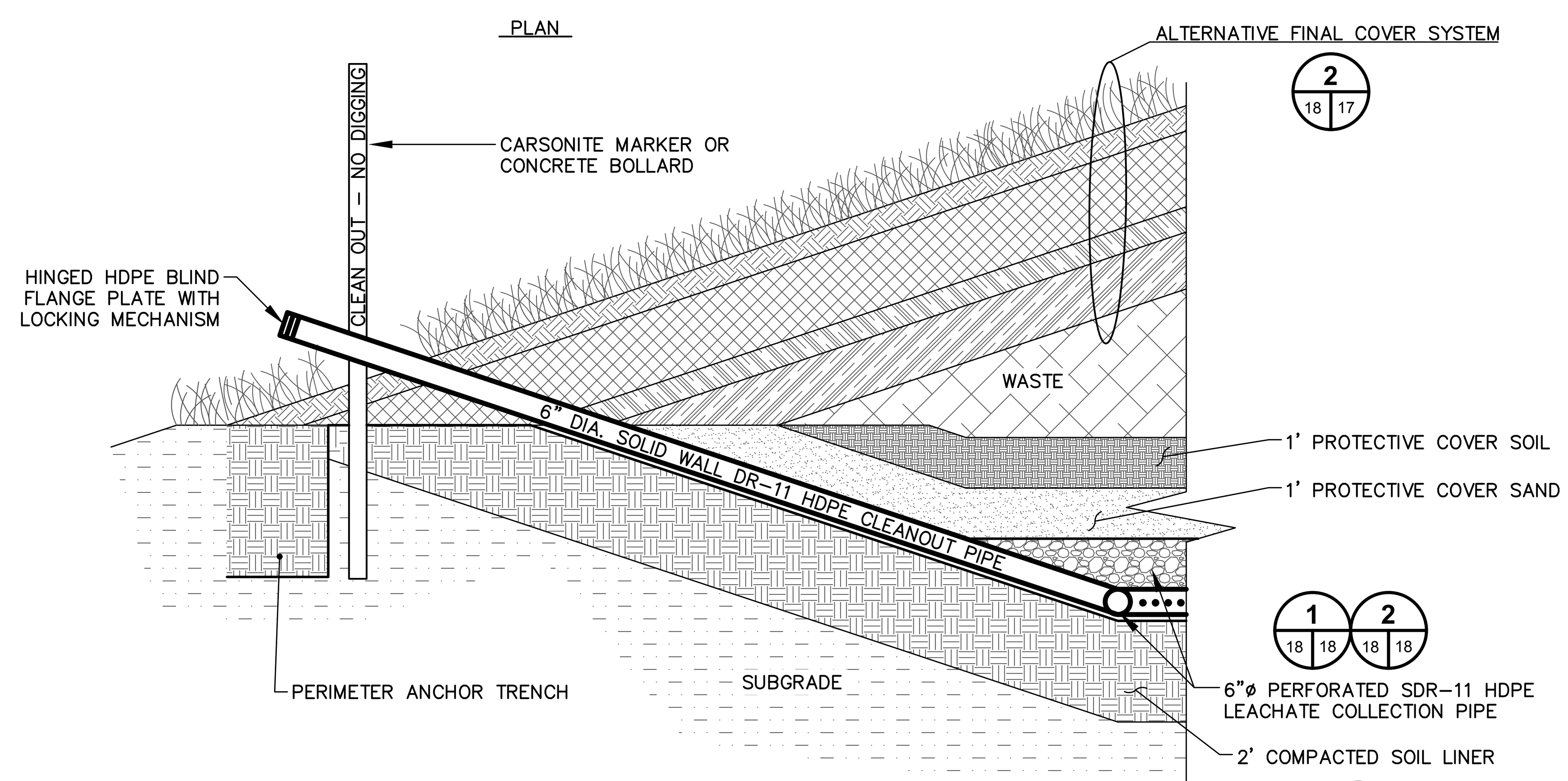
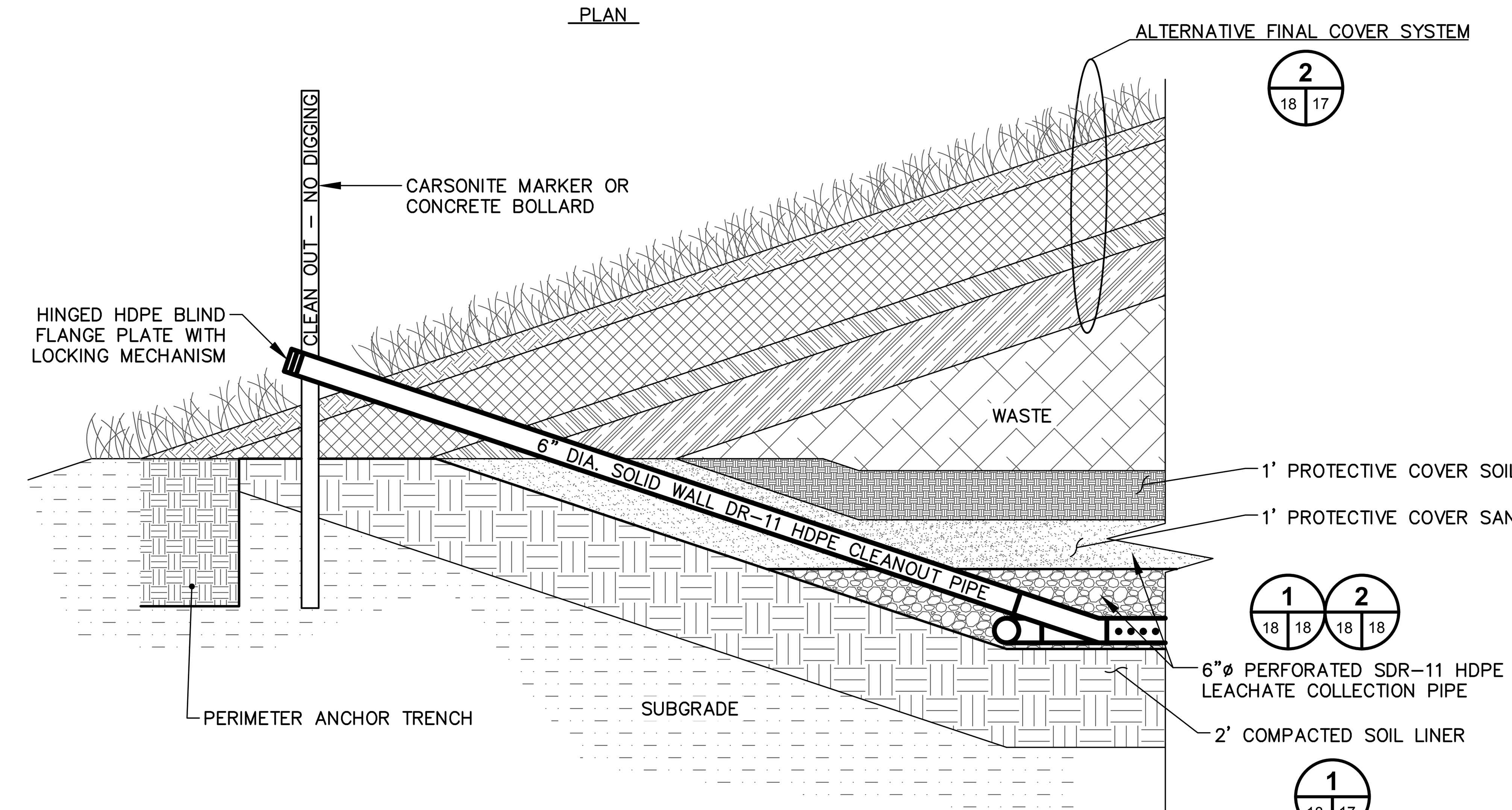


3 6 18 9 19 **LEACHATE COLLECTION PIPE CLEANOUT (TYP)**
NOT TO SCALE



3 6 18 9 19 **LEACHATE COLLECTION PIPE CLEANOUT (TYP)**
NOT TO SCALE

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REVISIONS

NO.	DATE	BY

ISSUED FOR REVIEW ____ DATE ____ BY

REVISIONS

NO.	COMMENTS	DATE	BY
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4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

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MICHAEL S. MAY
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
NO. 103104
STATE OF TENNESSEE

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EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

LEACHATE MANAGEMENT DETAILS 1

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
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Franklin, TN 37067-6209

P-18

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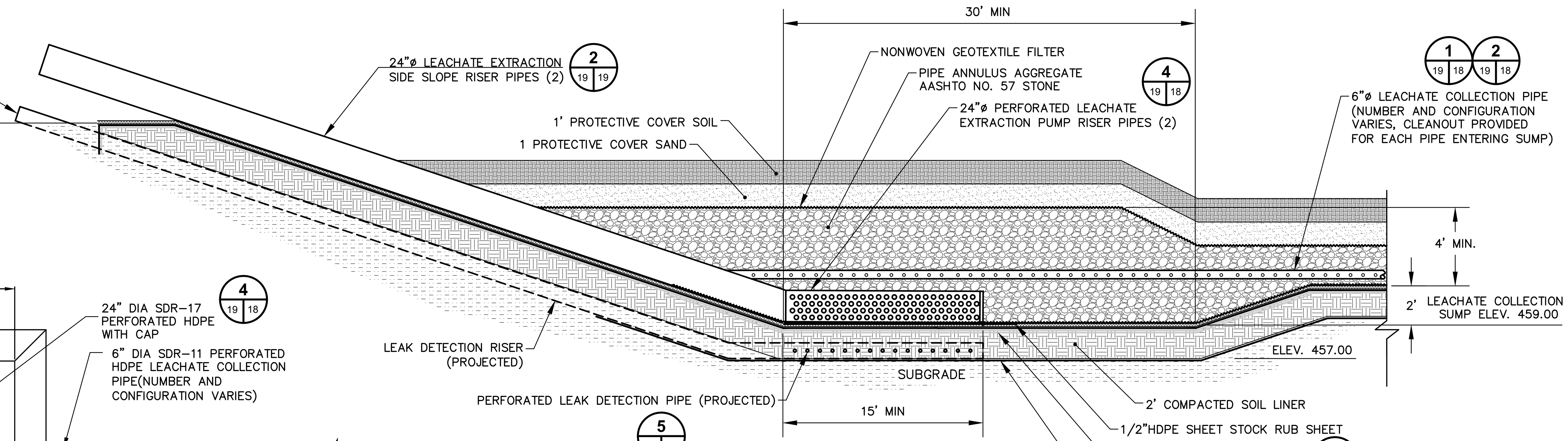
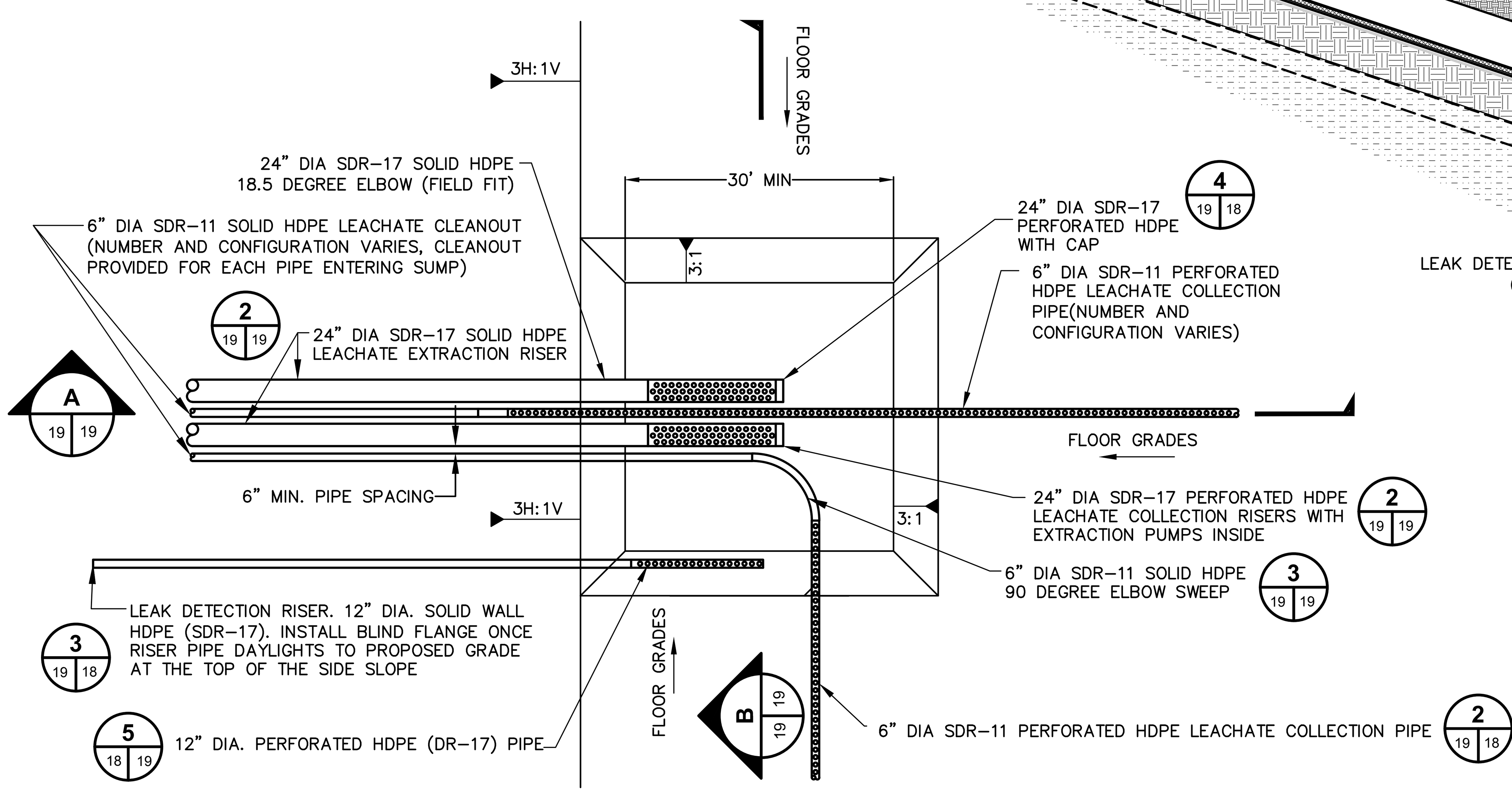


SECTION A

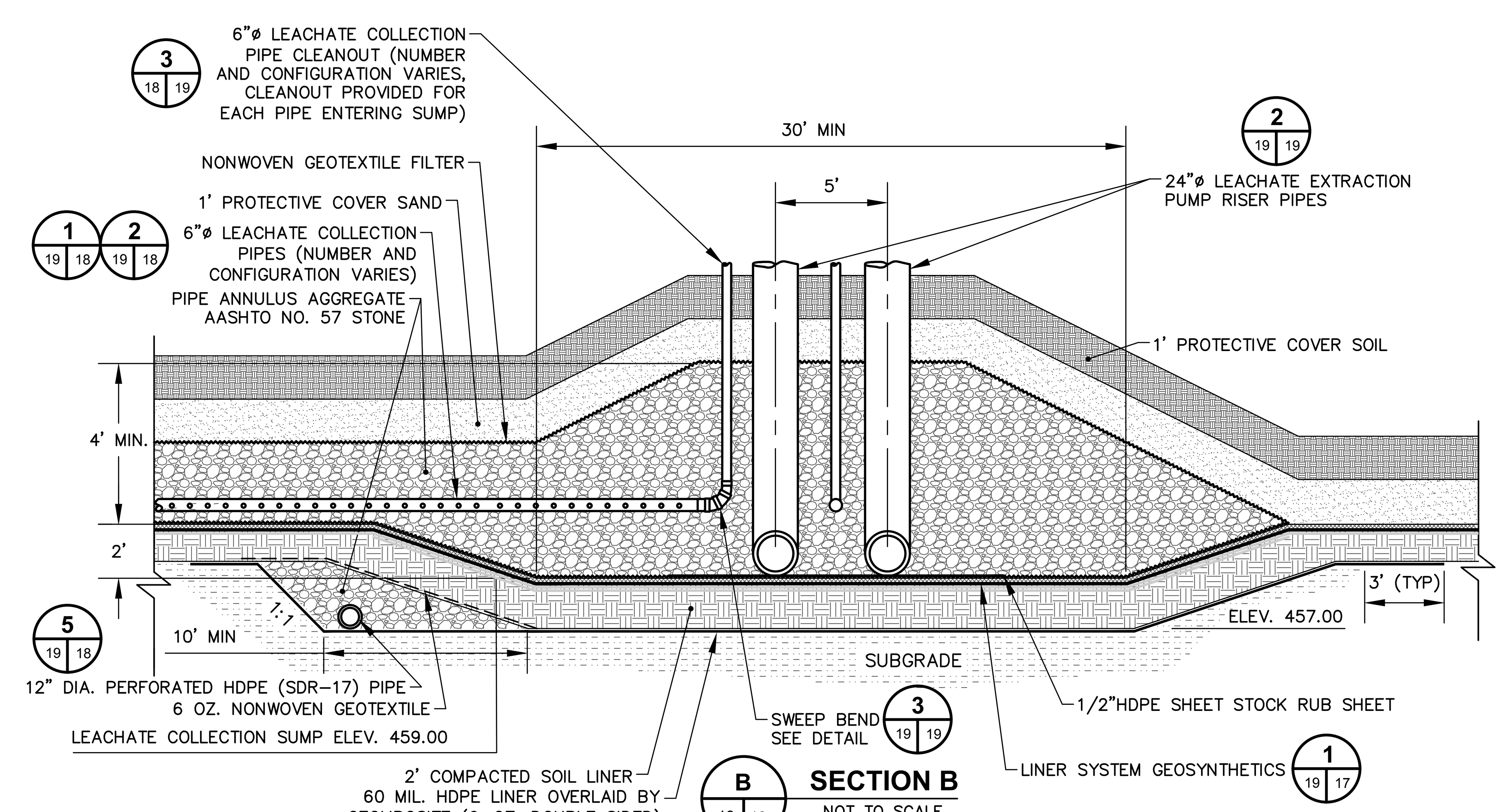
NOT TO SCALE

LEACHATE AGGREGATE CARBONATE CONTENT NOTE:
MAXIMUM CARBONATE CONTENT OF THE LEACHATE COLLECTION SYSTEM AGGREGATE SHALL BE 15%. CARBON CONTENT SHALL BE DETERMINED IN ACCORDANCE WITH ASTM-3042. ASTM-3042 SHALL BE CONDUCTED WITH NO pH MODIFICATION SOLUTION (6N HCL).



LEACHATE COLLECTION SUMP PLAN VIEW

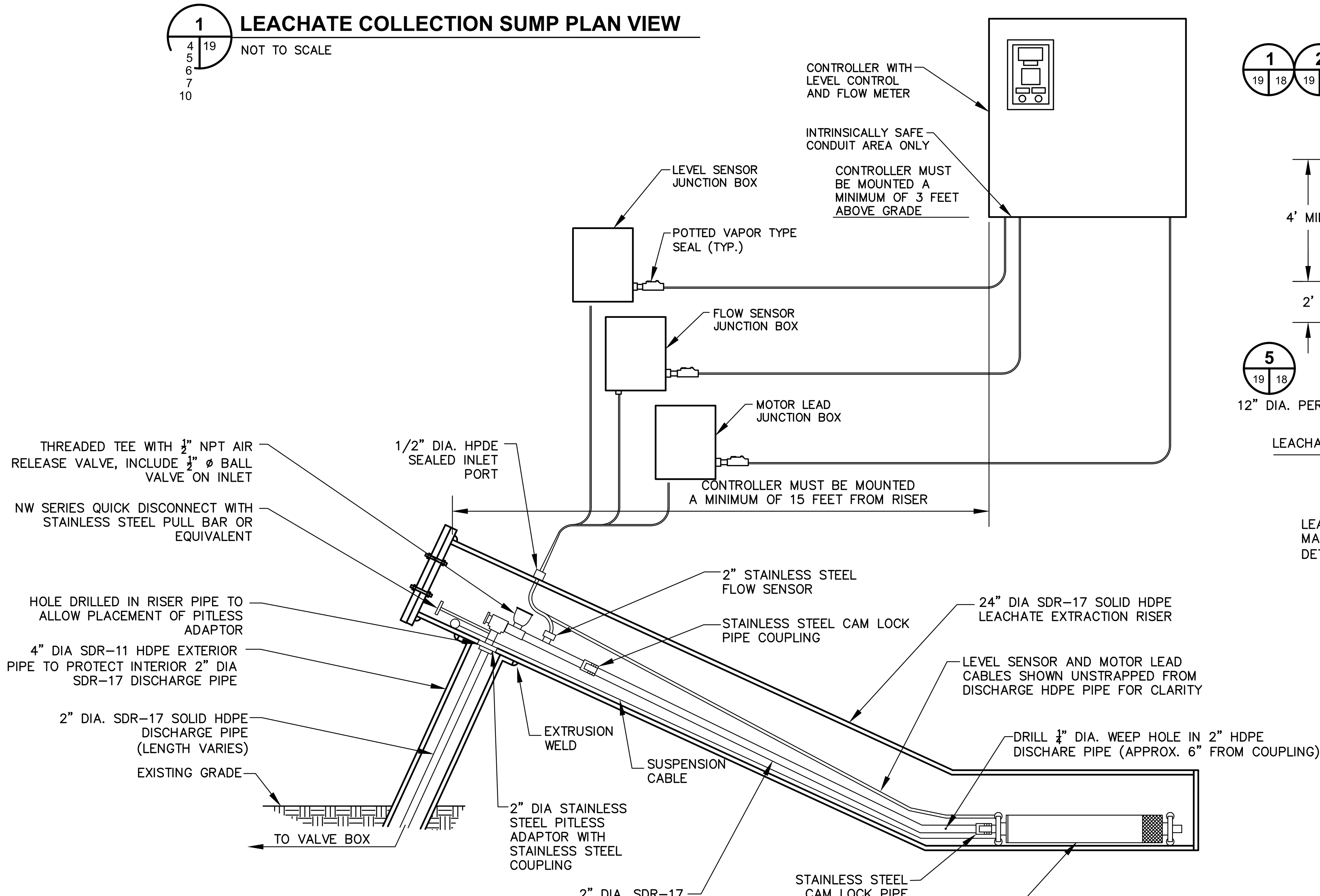
NOT TO SCALE



SECTION B

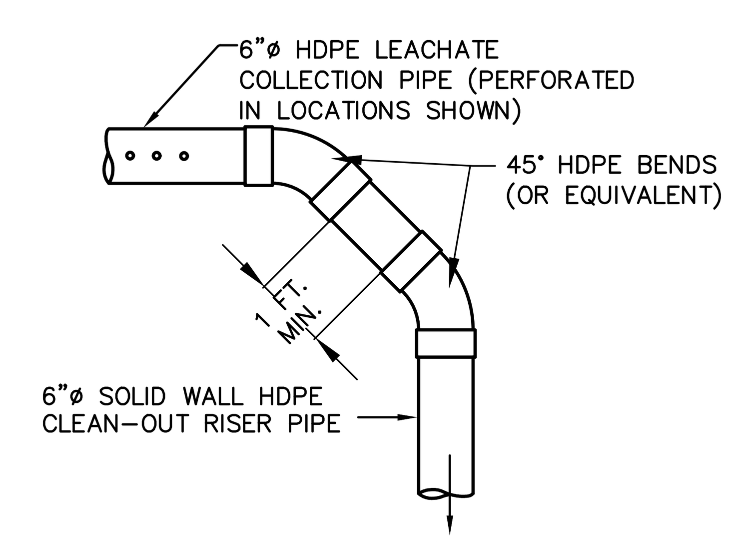
NOT TO SCALE

LEACHATE AGGREGATE CARBONATE CONTENT NOTE:
MAXIMUM CARBONATE CONTENT OF THE LEACHATE COLLECTION SYSTEM AGGREGATE SHALL BE 15%. CARBON CONTENT SHALL BE DETERMINED IN ACCORDANCE WITH ASTM-3042. ASTM-3042 SHALL BE CONDUCTED WITH NO pH MODIFICATION SOLUTION (6N HCL).



LEACHATE EXTRACTION PUMP SIDE SLOPE RISER

NOT TO SCALE



SWEEP BEND

NOT TO SCALE

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ADDENDUM NO.	ADDENDUM DATE	BY

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NO.	DATE	BY

ISSUED FOR REVIEW — DATE — BY

NO.	COMMENTS	DATE	BY
0	ISSUED FOR PERMITTING	7/1/16	NSP
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4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

RECORD DRAWINGS — DATE — BY

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MICHAEL S. MAY
REGISTERED ENGINEER
AGRICULTURE COMMERCE
NO. 103104
STATE OF TENNESSEE

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SCALE AS SHOWN	

EAST PHASE DISPOSAL FACILITY PERMIT DRAWINGS

LEACHATE MANAGEMENT DETAILS 2

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-19

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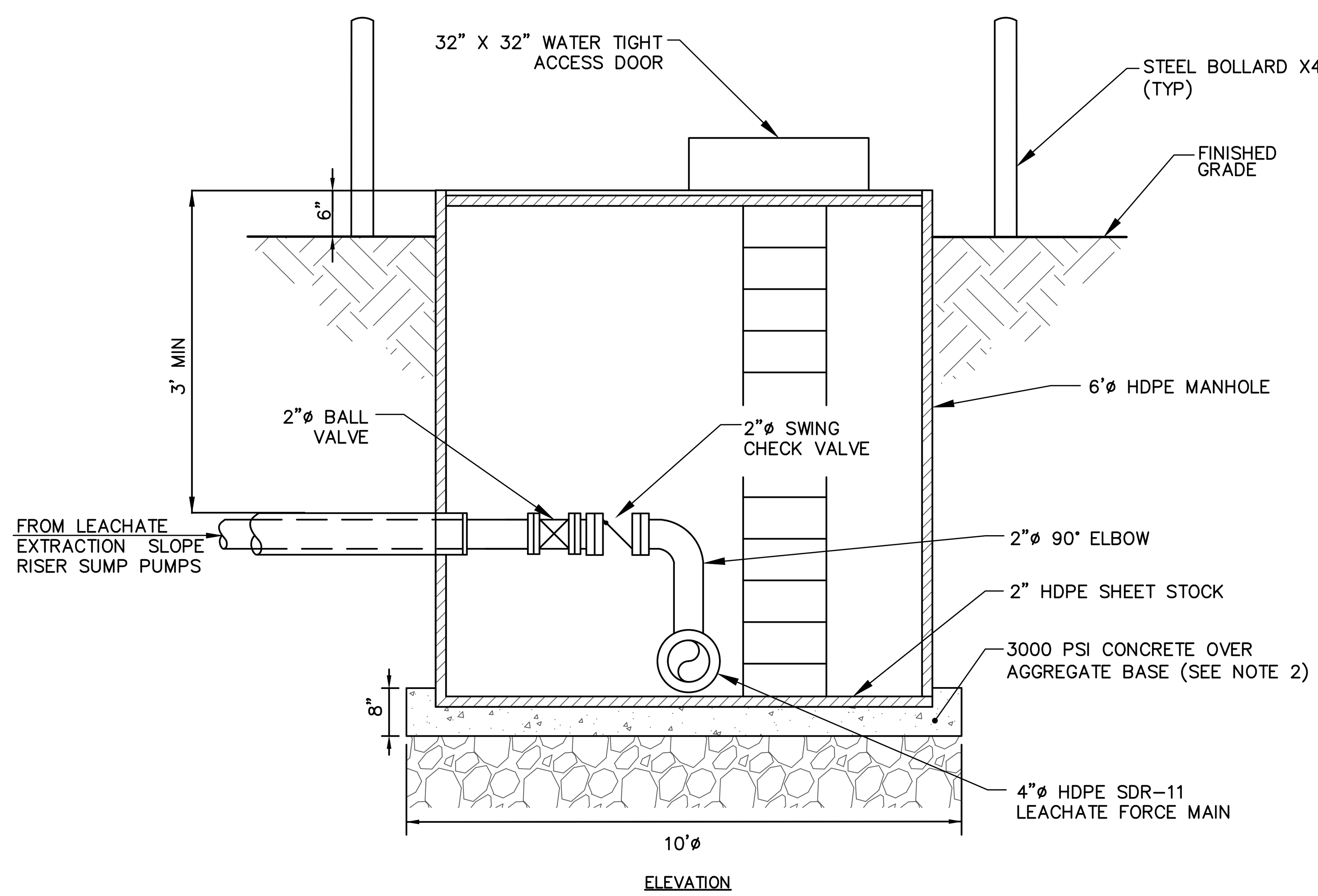
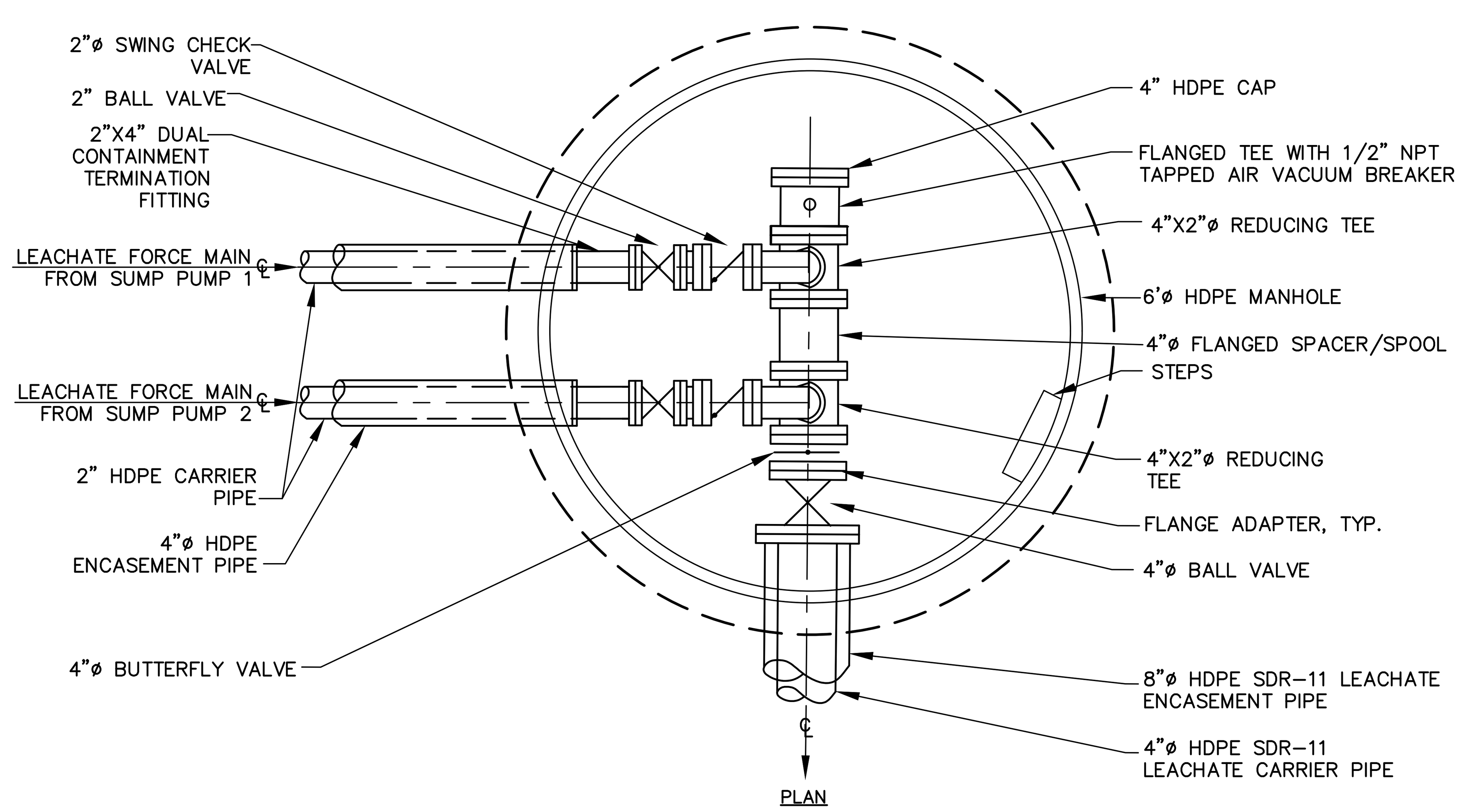
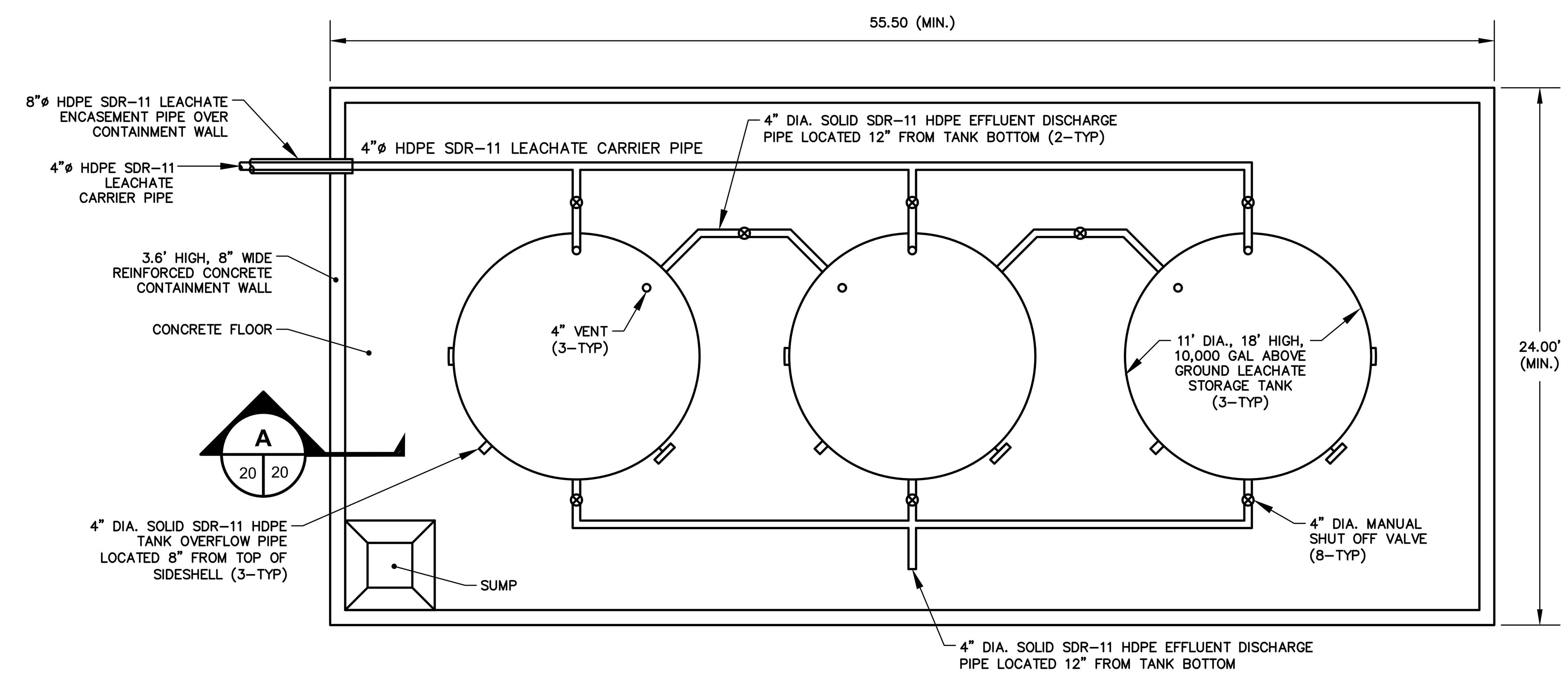




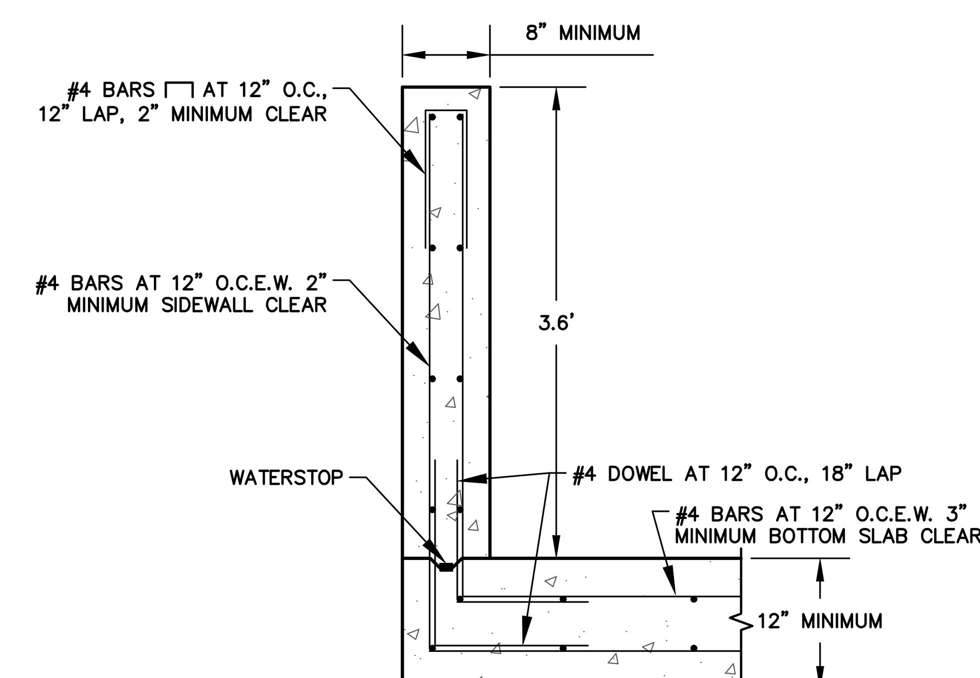
1 | 6 | 20
LEACHATE FORCE MAIN VALVE BOX
NOT TO SCALE

NOTES:

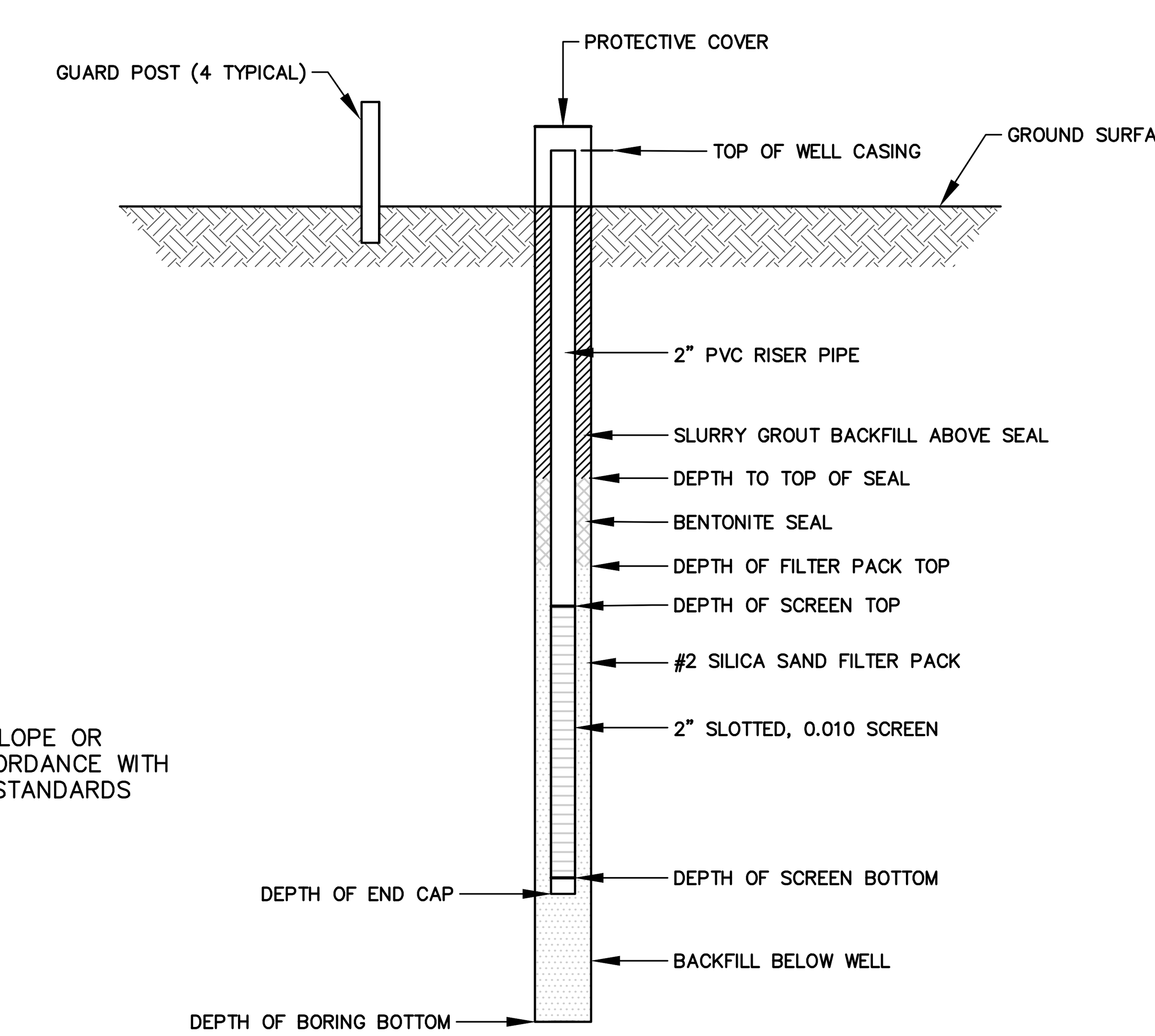
1. STEM EXTENSIONS TO BE PROVIDED WITH VALVES.
2. CONCRETE BASE TO BE UNDERLAIN BY 6" MIN OF AASHTO NO. 57 STONE OVER FILTER FABRIC ON A COMPACTED SOIL BASE.



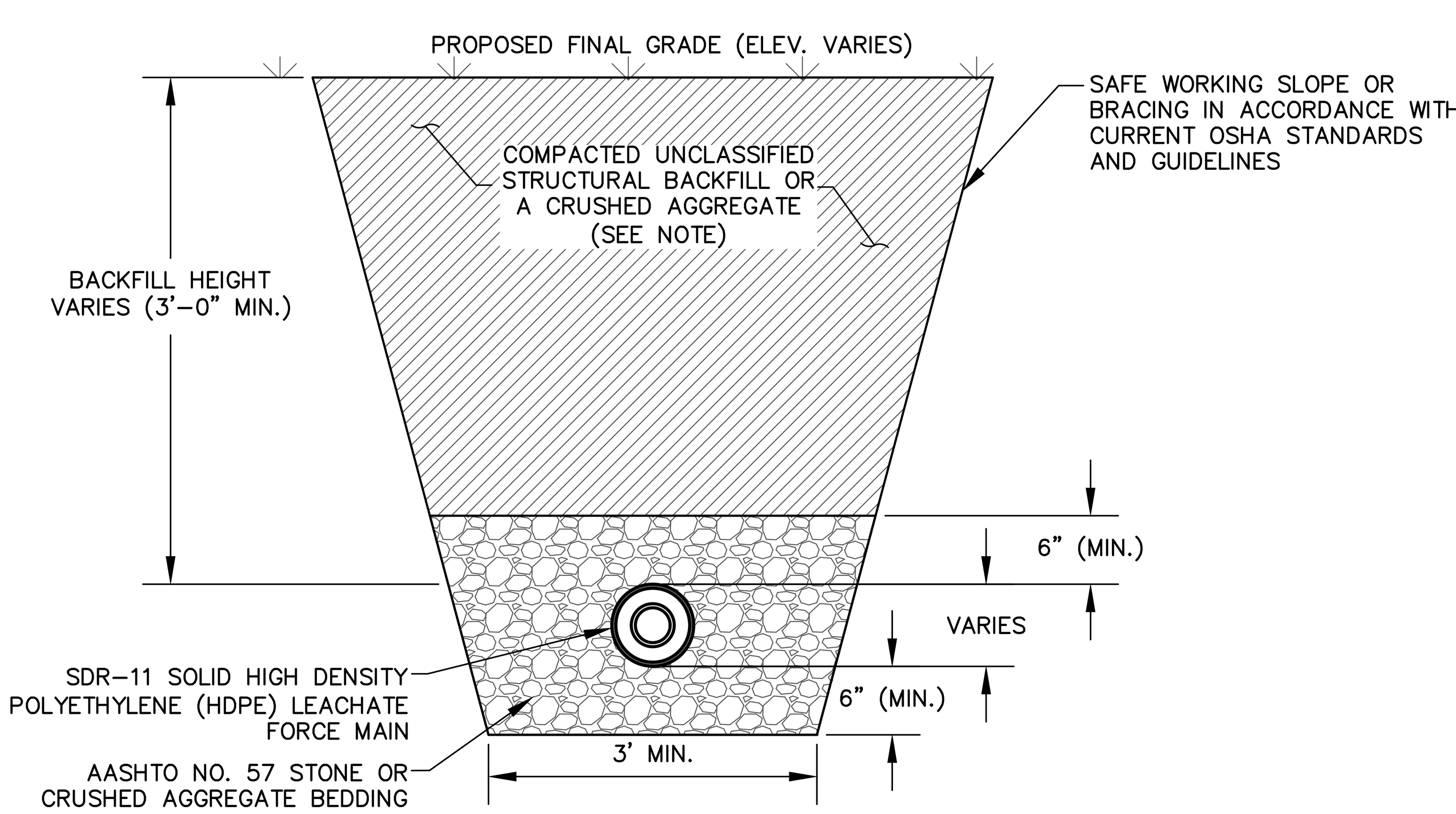
3 | 6 | 20
LEACHATE STORAGE TANKS
NOT TO SCALE



A | 20 | 20
SECTION A
NOT TO SCALE



4 | 4 | 20
MONITORING WELL DETAIL
NOT TO SCALE



2 | 6 | 20
LEACHATE FORCE MAIN TRENCH
NOT TO SCALE

NOTES:
CRUSHED AGGREGATE BACKFILL SHALL BE USED AT ALL LEACHATE FORCEMAIN ROAD CROSSINGS.

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ADDENDUM NO.	ADDENDUM DATE	BY

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REVISIONS		
NO.	DATE	BY

ISSUED FOR REVIEW DATE BY

REVISIONS			
NO.	COMMENTS	DATE	BY
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5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

RECORD DRAWINGS DATE BY

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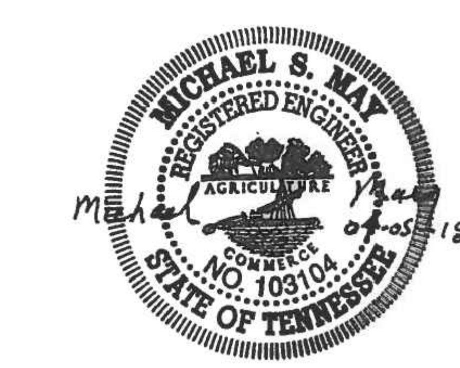

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EAST PHASE DISPOSAL FACILITY
PERMIT DRAWINGS

LEACHATE MANAGEMENT DETAILS 3

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-20

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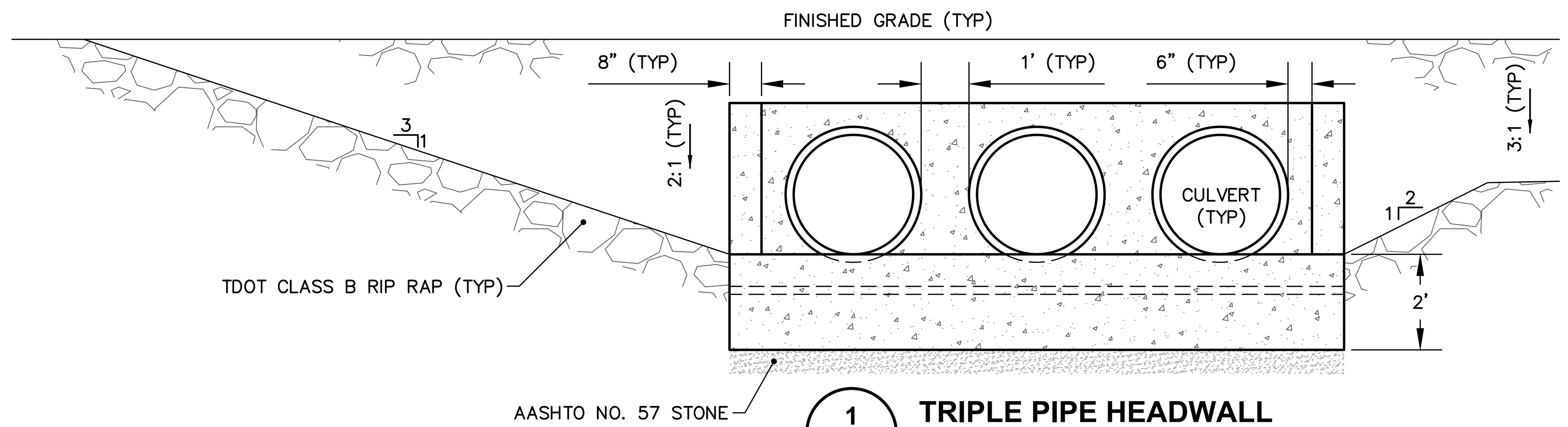
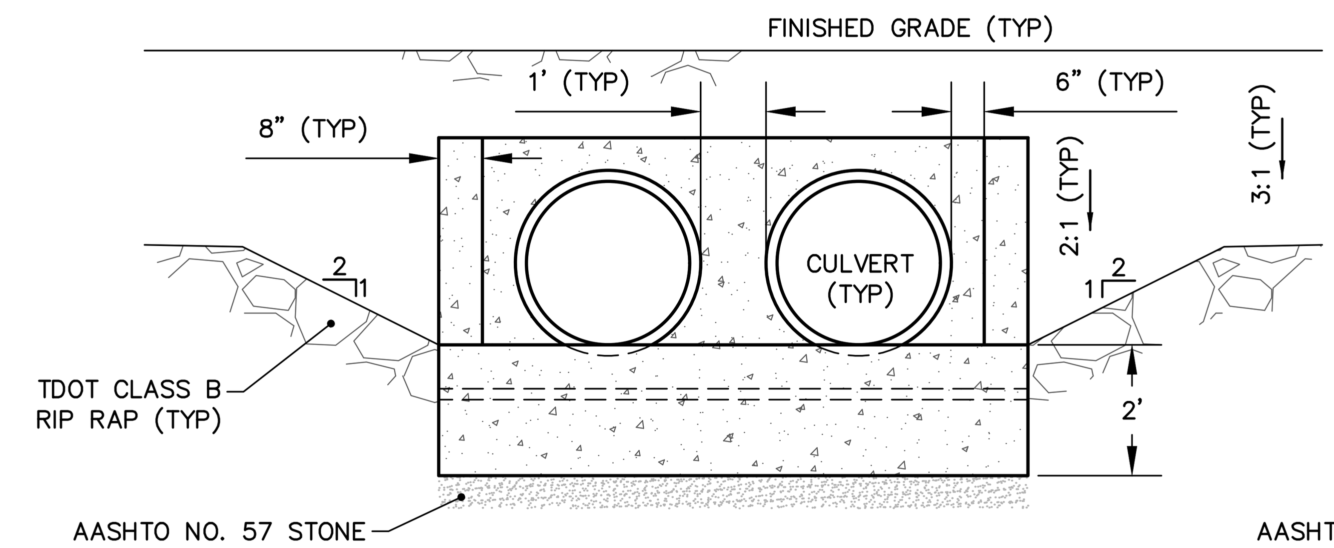
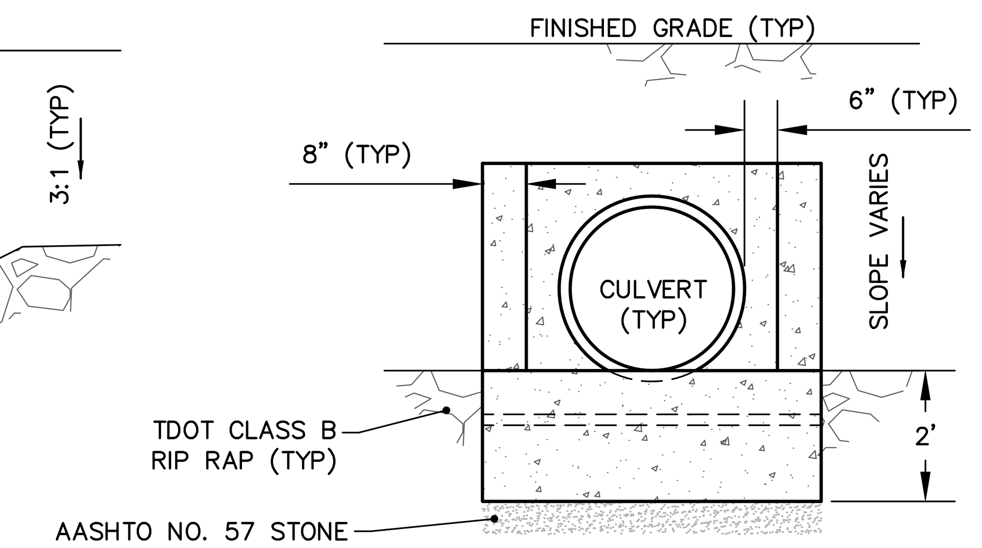


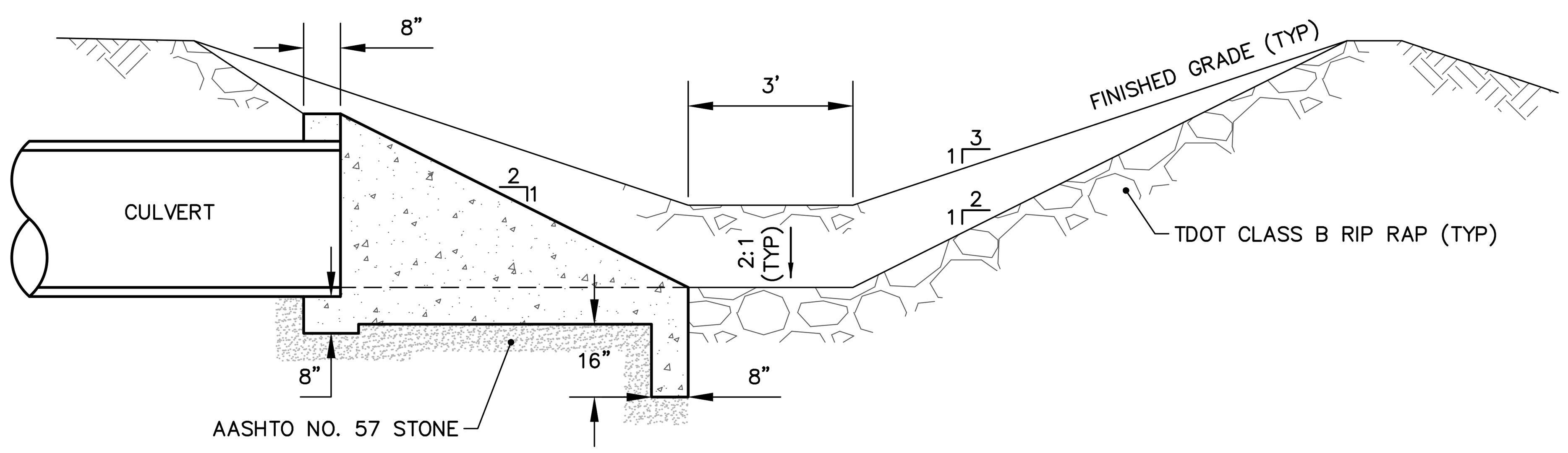
1 TRIPLE PIPE HEADWALL (14 | 21)
NOT TO SCALE



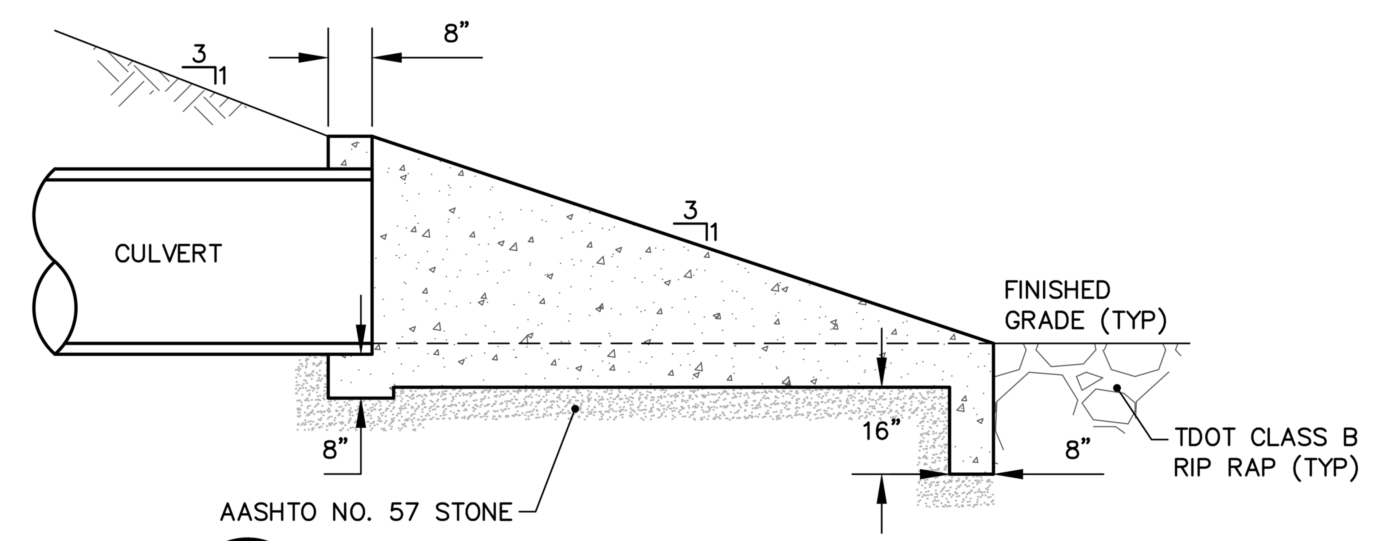
2 DOUBLE PIPE HEADWALL (10, 14 | 21)
NOT TO SCALE



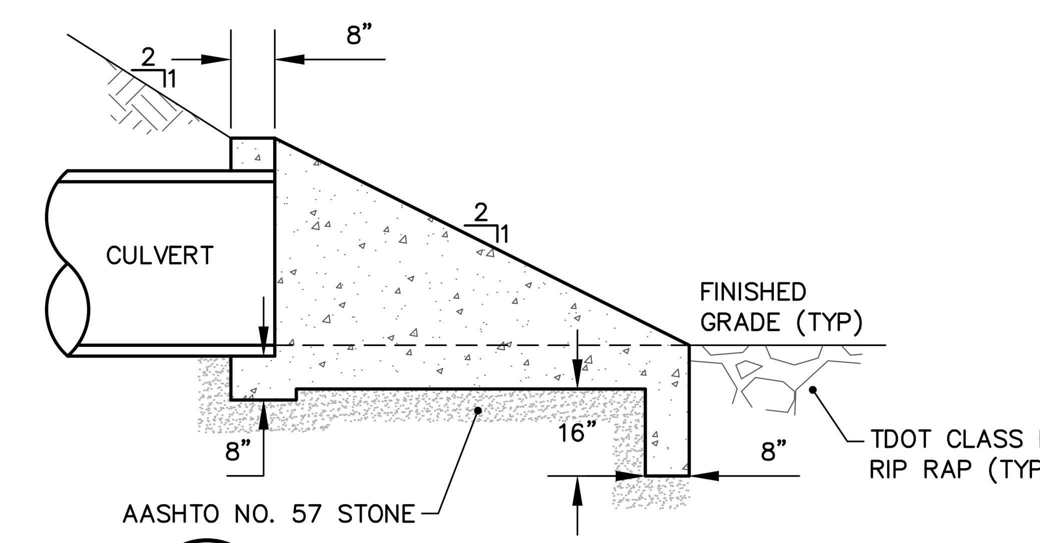
3 SINGLE PIPE HEADWALL (14 | 21)
NOT TO SCALE



4 IMPROVED INLET HEADWALL PROFILE (14 | 21)
NOT TO SCALE

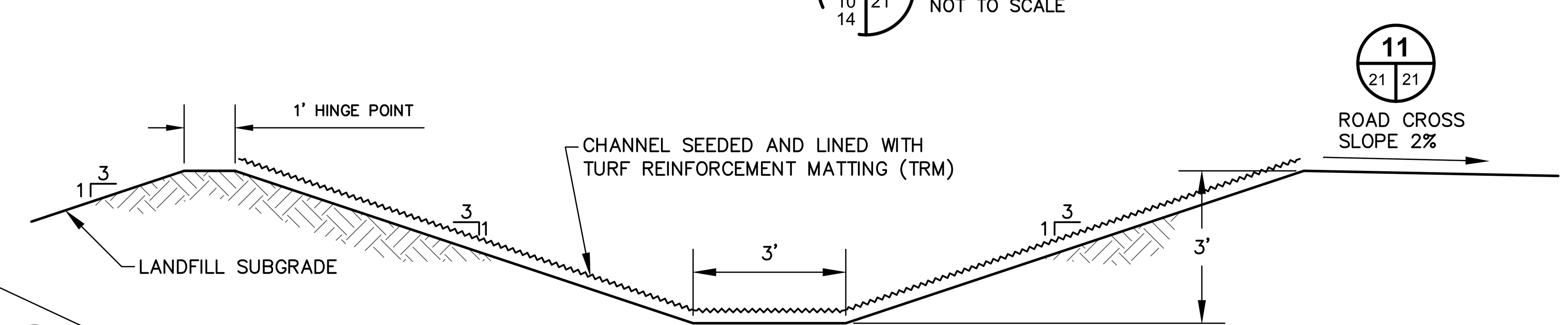


5 3:1 HEADWALL PROFILE (10, 14 | 21)
NOT TO SCALE



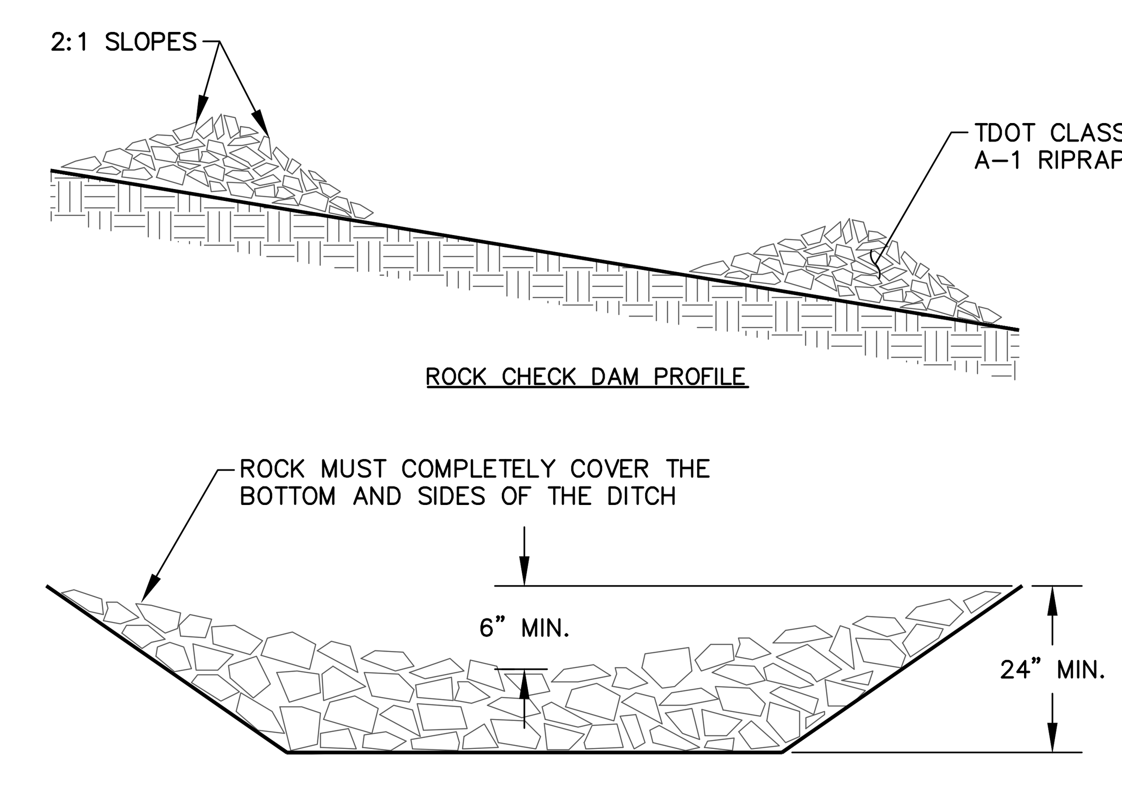
6 2:1 HEADWALL PROFILE (14 | 21)
NOT TO SCALE

NOTE:
RIP RAP MAY BE USED AS AN ALTERNATIVE TO ALL CONCRETE HEADWALLS AND ENDWALLS DURING CONSTRUCTION.



10 PERIMETER DITCH (10, 11, 12, 13, 14, 16, 17, 21 | 21)
NOT TO SCALE

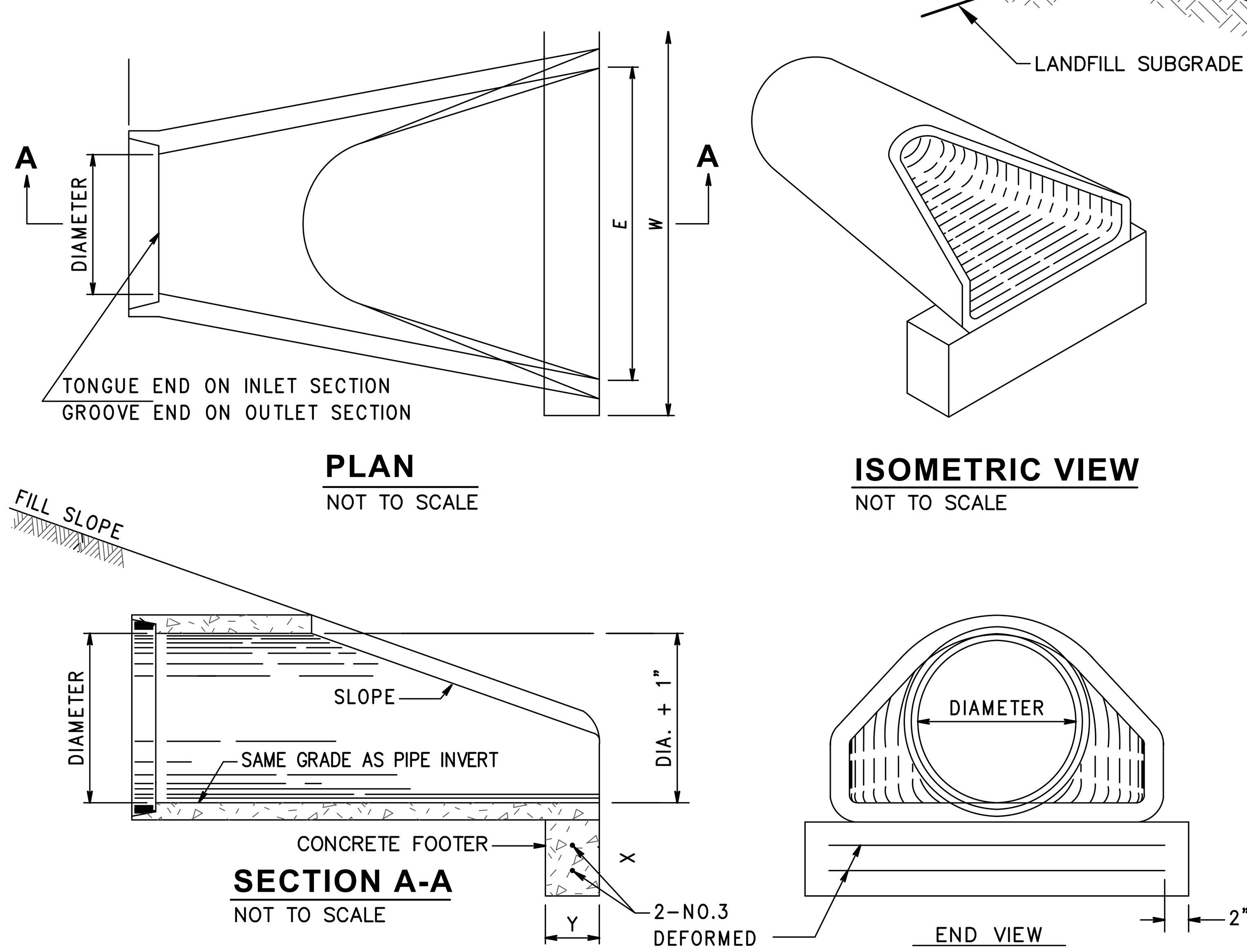
NOTE:
TURF REINFORCEMENT MATTING SHALL BE GREENARMOR 7010 OR APPROVED EQUIVALENT.



9 ROCK CHECK DAM DETAIL (10, 11, 12, 13, 14 | 21)
NOT TO SCALE

NOTES:

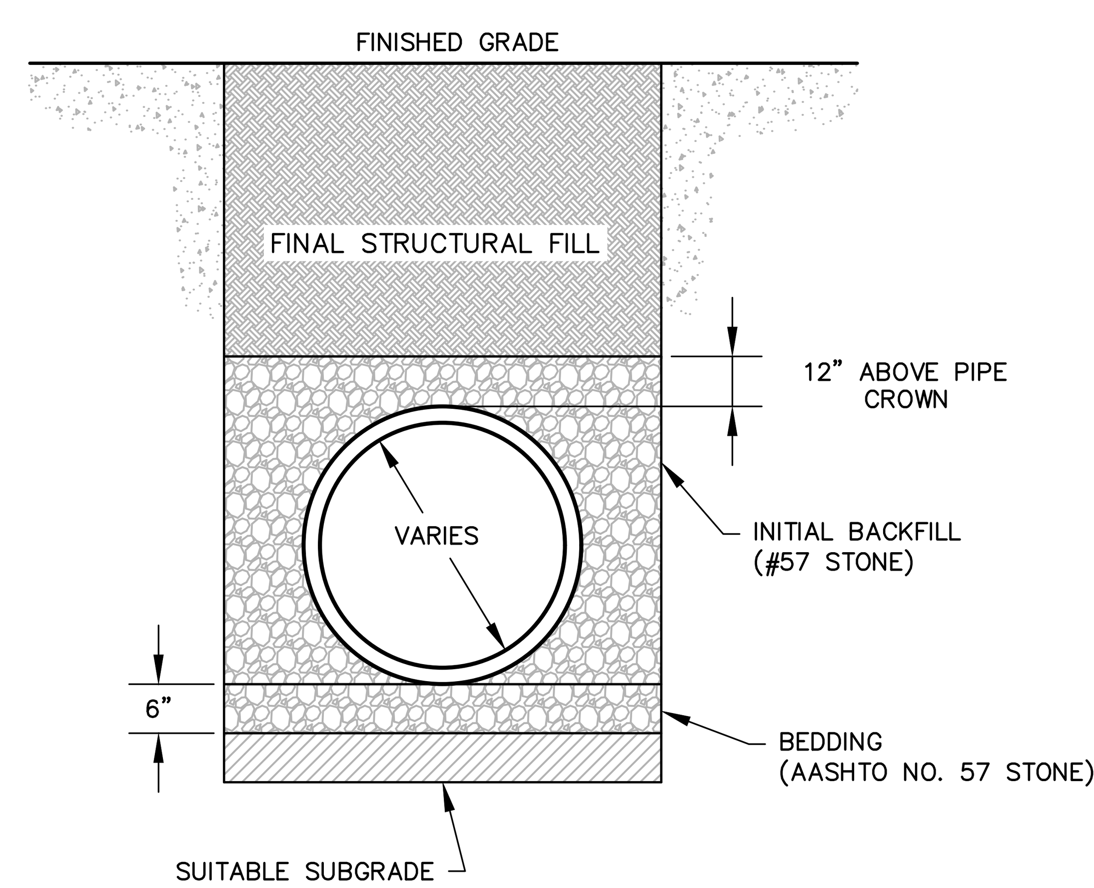
1. ROCK CHECK DAMS TO BE UNDERLAIN BY A 10 OZ/SQ. YD. NONWOVEN GEOTEXTILE.
2. ROCK CHECK DAMS TO BE INSTALLED AS NEEDED AROUND LANDFILL CONSTRUCTION AREAS.



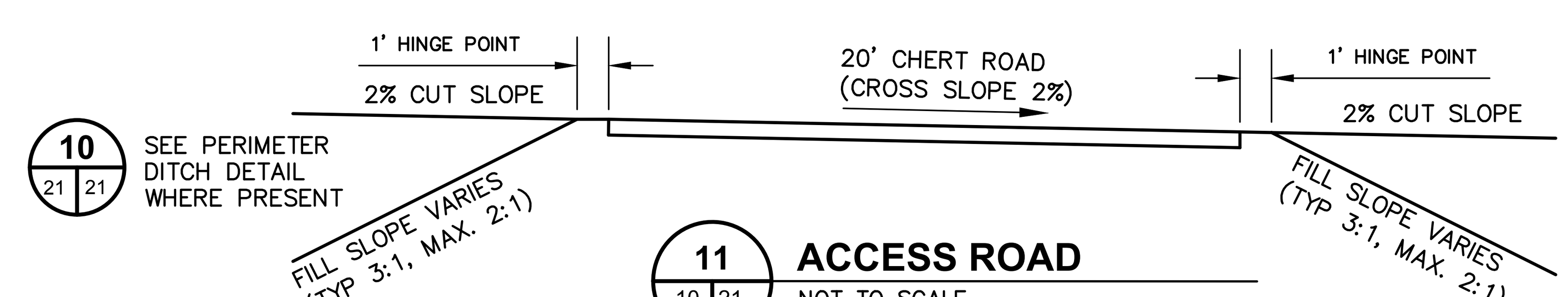
7 FLARED END SECTION (14 | 21)
NOT TO SCALE

NOTES:

1. PRE-CAST END SECTIONS ARE TO BE PROVIDED ON OPEN ENDS OF REINFORCED CONCRETE PIPE CULVERTS AS SHOWN IN THE CULVERT SCHEDULE.
2. FINAL DIMENSIONS AND REINFORCEMENT TO BE DETERMINED BASED ON PIPE DIAMETER BY PREFAB CONTRACTOR.



8 TYPICAL CULVERT BEDDING (10, 14 | 21)
NOT TO SCALE



11 ACCESS ROAD (10, 11, 12, 13, 14, 16, 17, 21 | 21)
NOT TO SCALE

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REVISIONS

NO.	DATE	BY

ISSUED FOR REVIEW — DATE — BY

REVISIONS

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0	ISSUED FOR PERMITTING	7/1/16	NSP
1	LEACHATE TANK, ROAD SECTION	8/10/16	NSP
2	TDEC PERMIT REVIEW COMMENTS	1/10/17	NSP
3	TDEC PERMIT REVIEW COMMENTS	5/9/17	NSP
4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

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STORMWATER MANAGEMENT DETAILS 1

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-21

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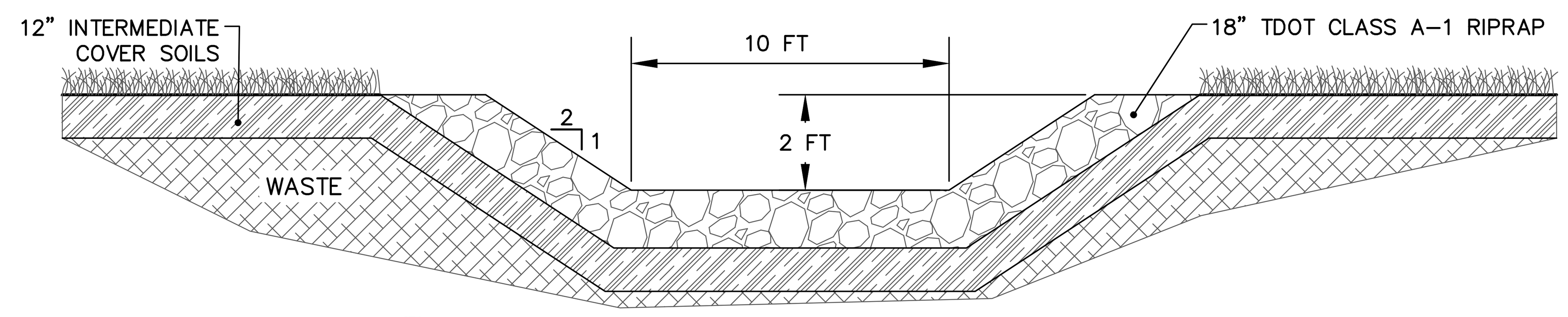
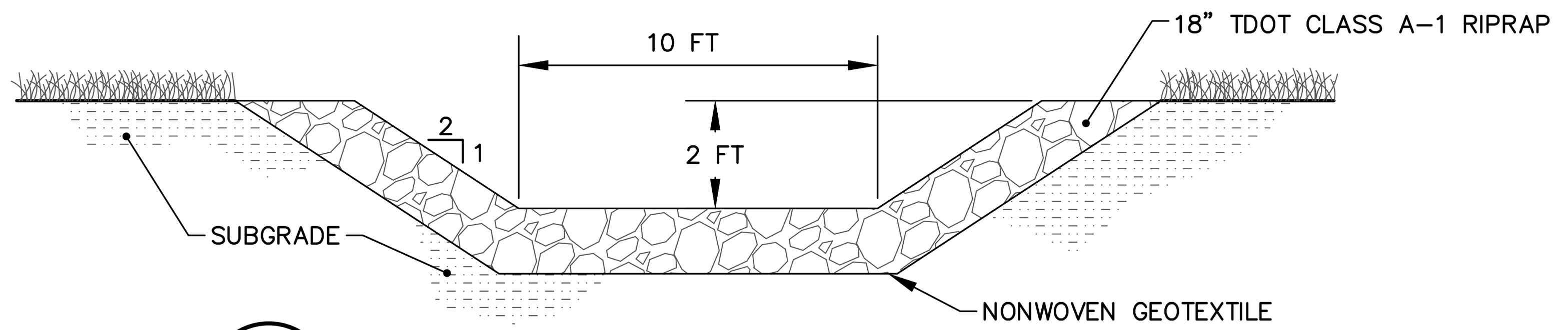
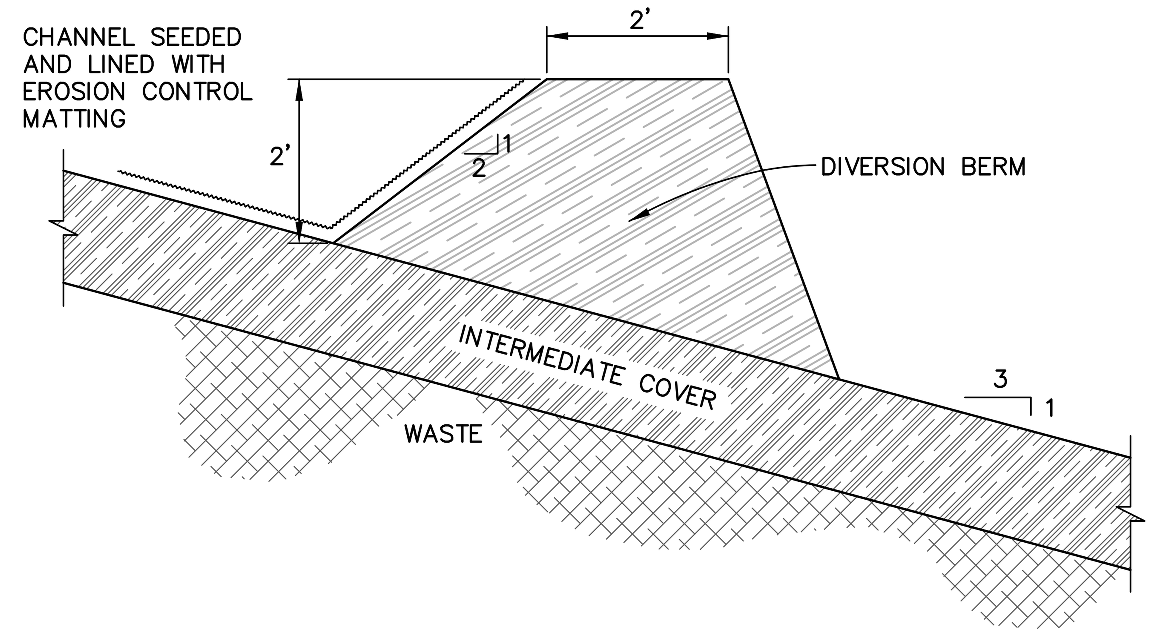


1 OPERATIONAL RIP RAP LINED LETDOWN
8 | 22
11
12
13
NOT TO SCALE



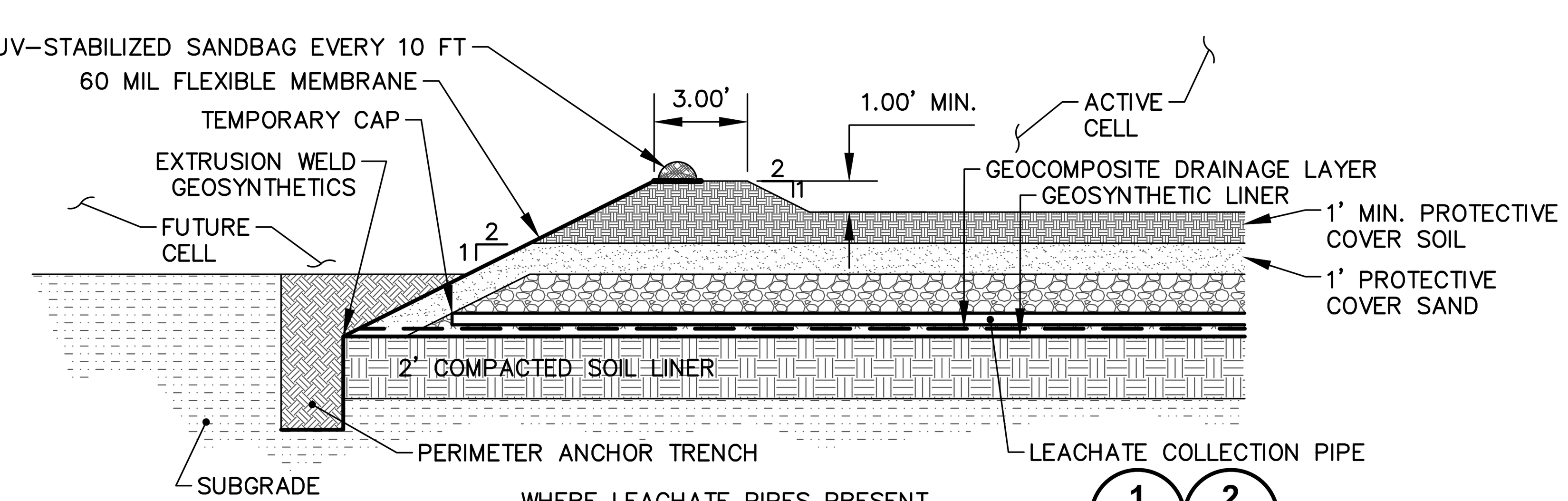
6 STORMWATER RIP RAP LINED LETDOWN
10 | 22
NOT TO SCALE

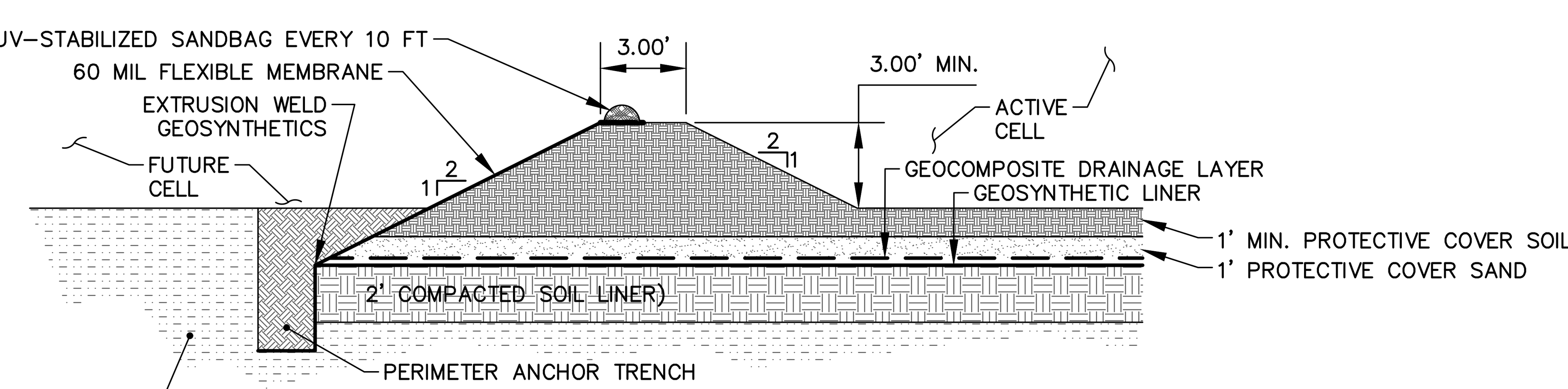


3 TACK-ON STORM WATER DIVERSION BERM
11 | 22
12
13
NOT TO SCALE

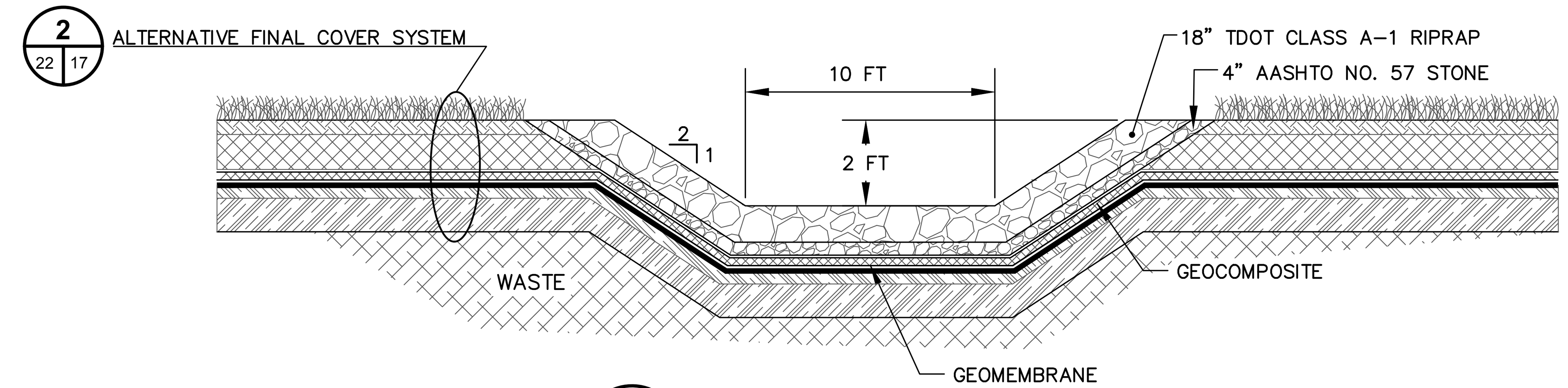
NOTES:

1. LOCATE ON INTERMEDIATE SLOPES AS NEEDED DURING OPERATIONS.

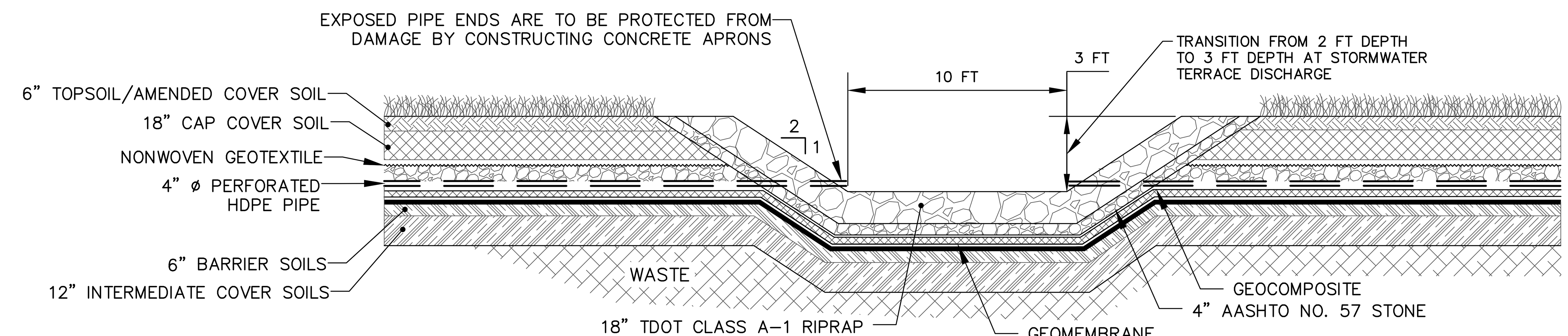




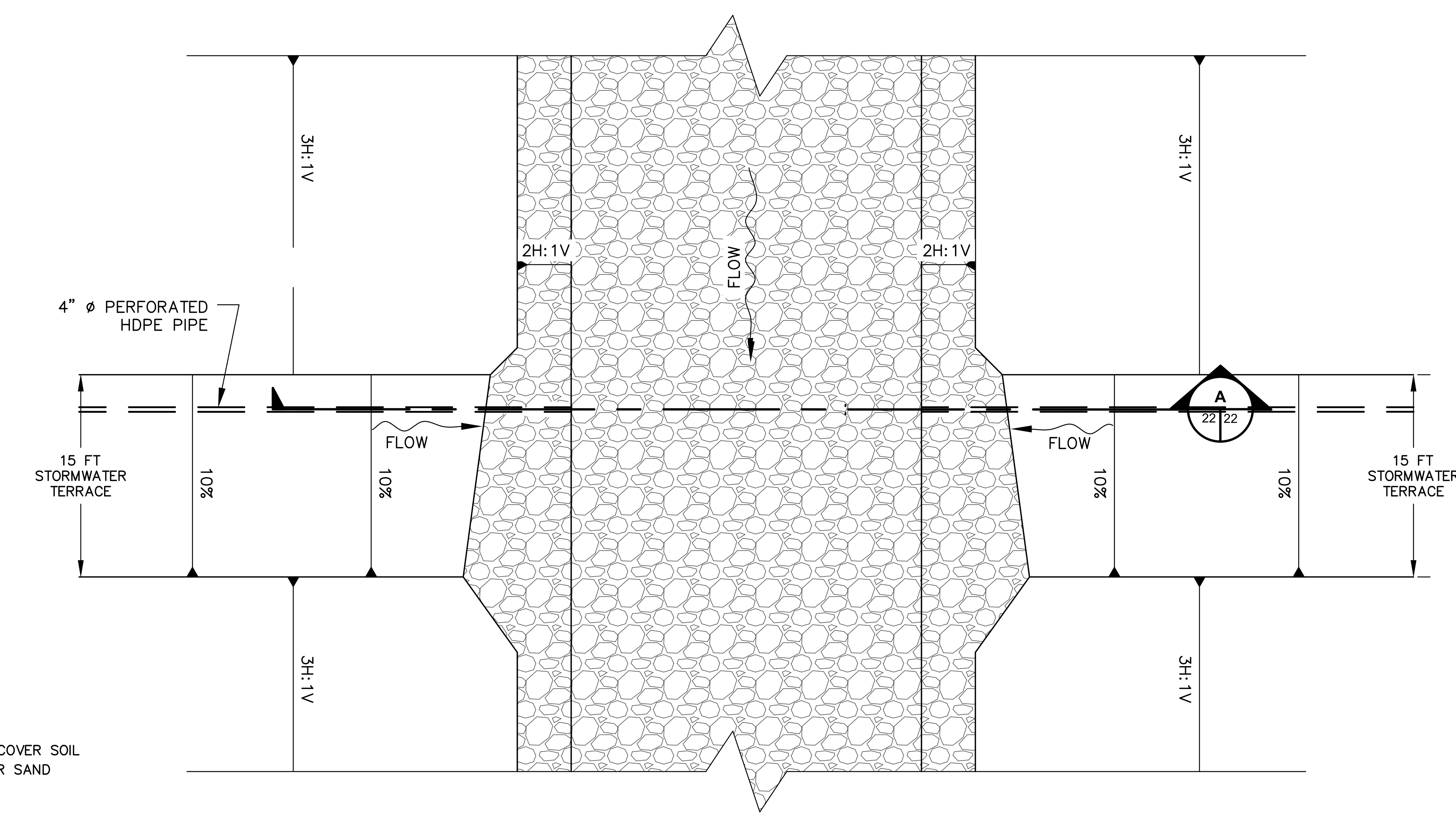
4 CELL DIVISION BERM
10 | 22
11
12
NOT TO SCALE



2 PERMANENT RIP RAP LINED LETDOWN
9 | 22
14
NOT TO SCALE



A SECTION A-A'
22 | 22
NOT TO SCALE



5 STORM WATER TERRACE DISCHARGE TO ROCK LETDOWN
14 | 22
NOT TO SCALE

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4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

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STORMWATER MANAGEMENT DETAILS 2

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

P-22

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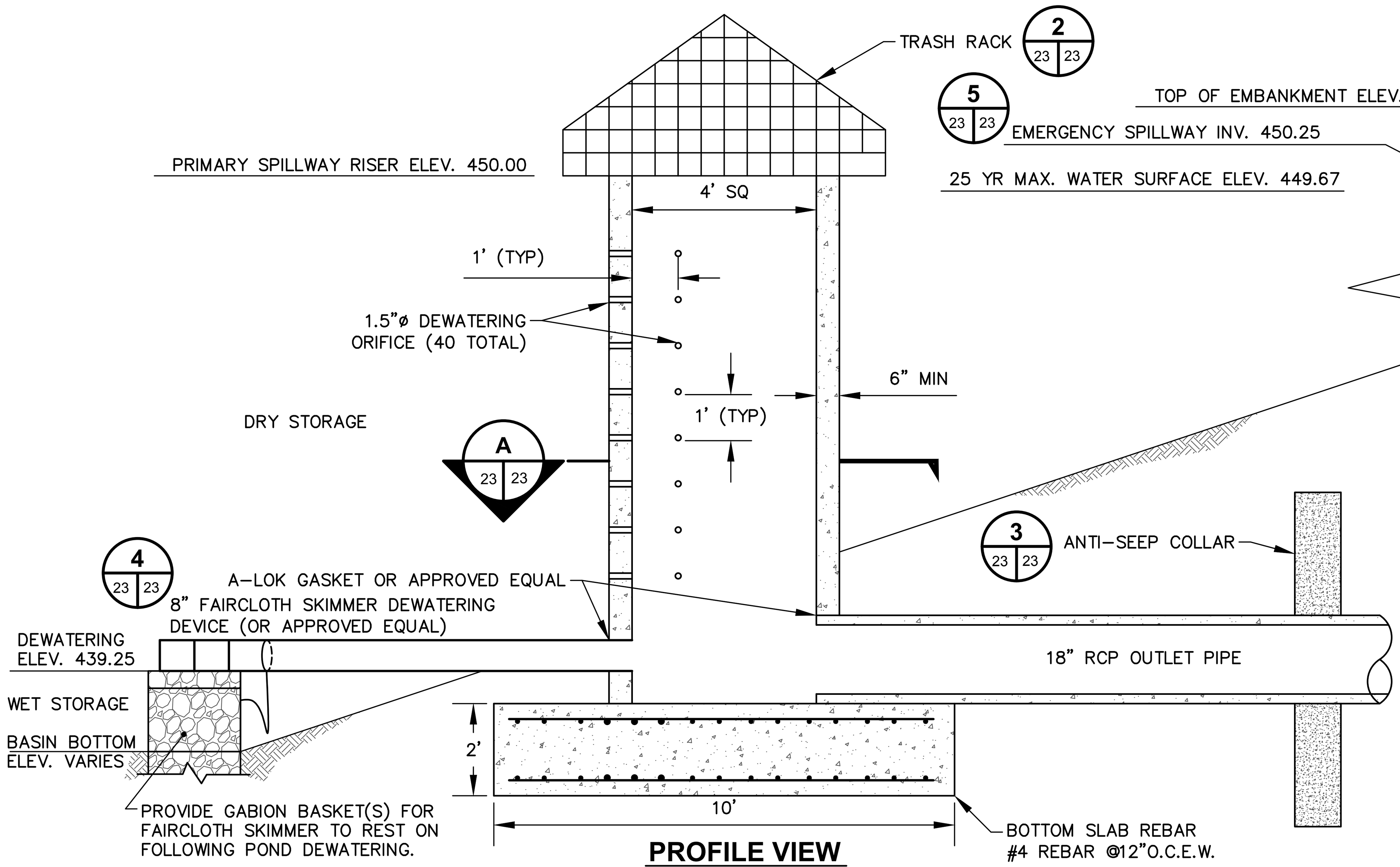
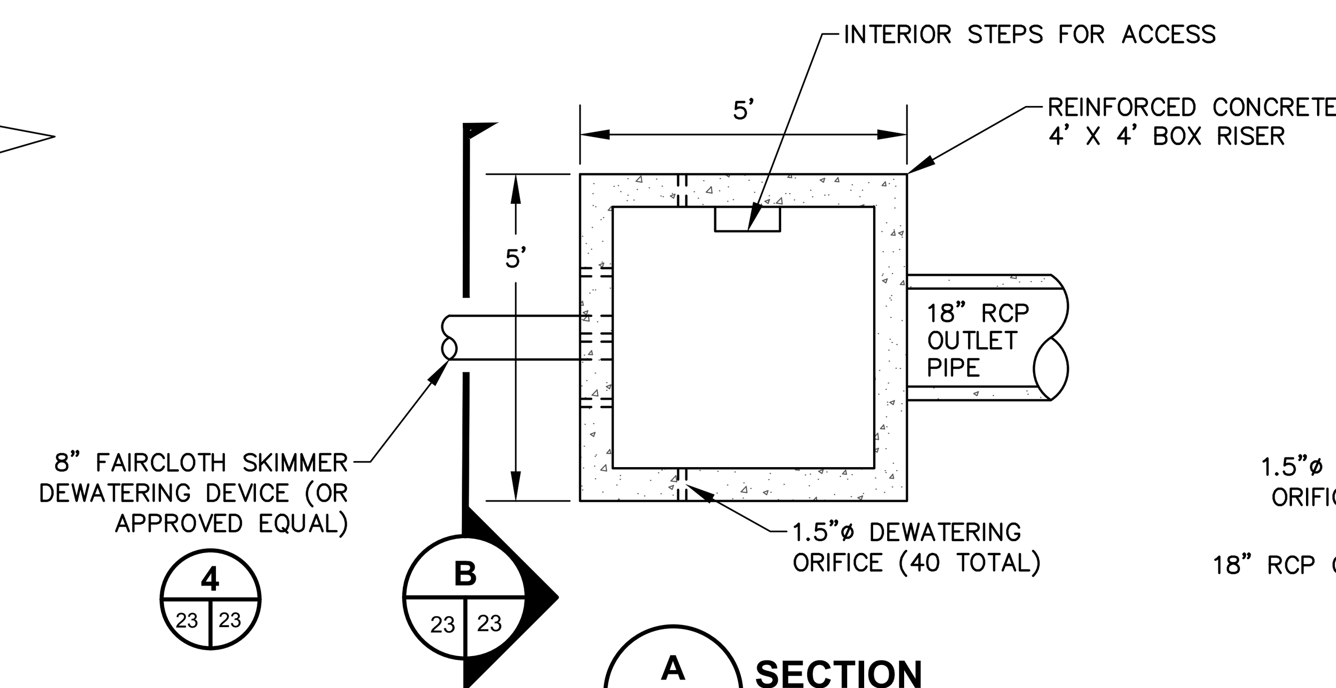


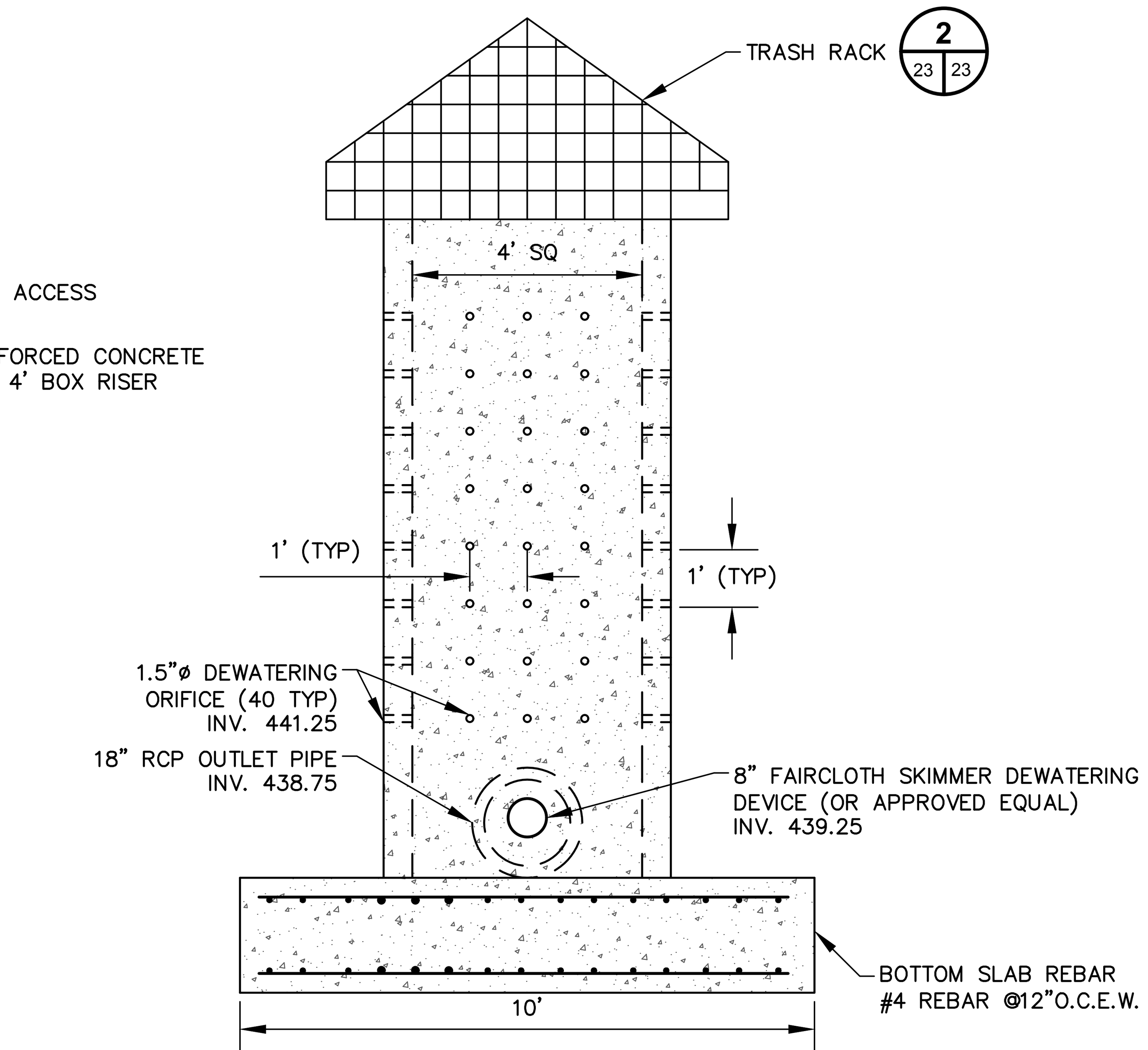
1 14 15 23 **SEDIMENT BASIN OUTLET STRUCTURE**
NOT TO SCALE

NOTES:

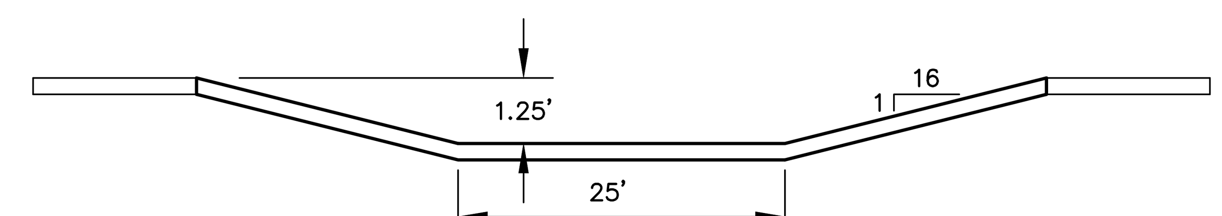
1. PRE-CAST RISER SHALL BE MINIMUM 3000 PSI CONCRETE.
2. WALL THICKNESS SHOWN AS CONCEPTUAL. FINAL WALL THICKNESS AND REINFORCEMENT TO BE DETERMINED BY THE PREFAB CONTRACTOR.
3. BOTTOM SLAB MAY BE PRE-CAST AS PART OF THE BOX RISER STRUCTURE OR MAY BE CAST-IN-PLACE.
4. BUTYL RUBBER WATER TIGHT SEAL AT JOINTS.
5. GABION BASKET(S) SHALL FORM AN ARC AROUND THE SEDIMENT BASIN PRIMARY SPILLWAY STRUCTURE, TIE INTO BASIN SIDE SLOPES, AND INSTALLED PER MANUFACTURE'S RECOMMENDATION.
6. GABION BASKET(S) SHALL INCLUDE A 1" METAL BAND AROUND THE GABION BASKET(S)' PERIMETER SECURELY ATTACHED AND SET TO ELEVATION 438.00. THE METAL BAND SHALL BE POWDERED COATED WITH HIGH VISIBILITY CONSTRUCTION ORANGE.
7. THE FAIRCLOTH SKIMMER SHALL INCLUDE A TETHER SYSTEM TO ASSURE THE RESTING POSITION REMAIN ON THE GABION BASKET(S) WHILE PROVIDING THE NECESSARY RANGE OF MOTION TO REACH A WATER SURFACE ELEVATION OF 450.00.



A 23 23 **SECTION**
NOT TO SCALE



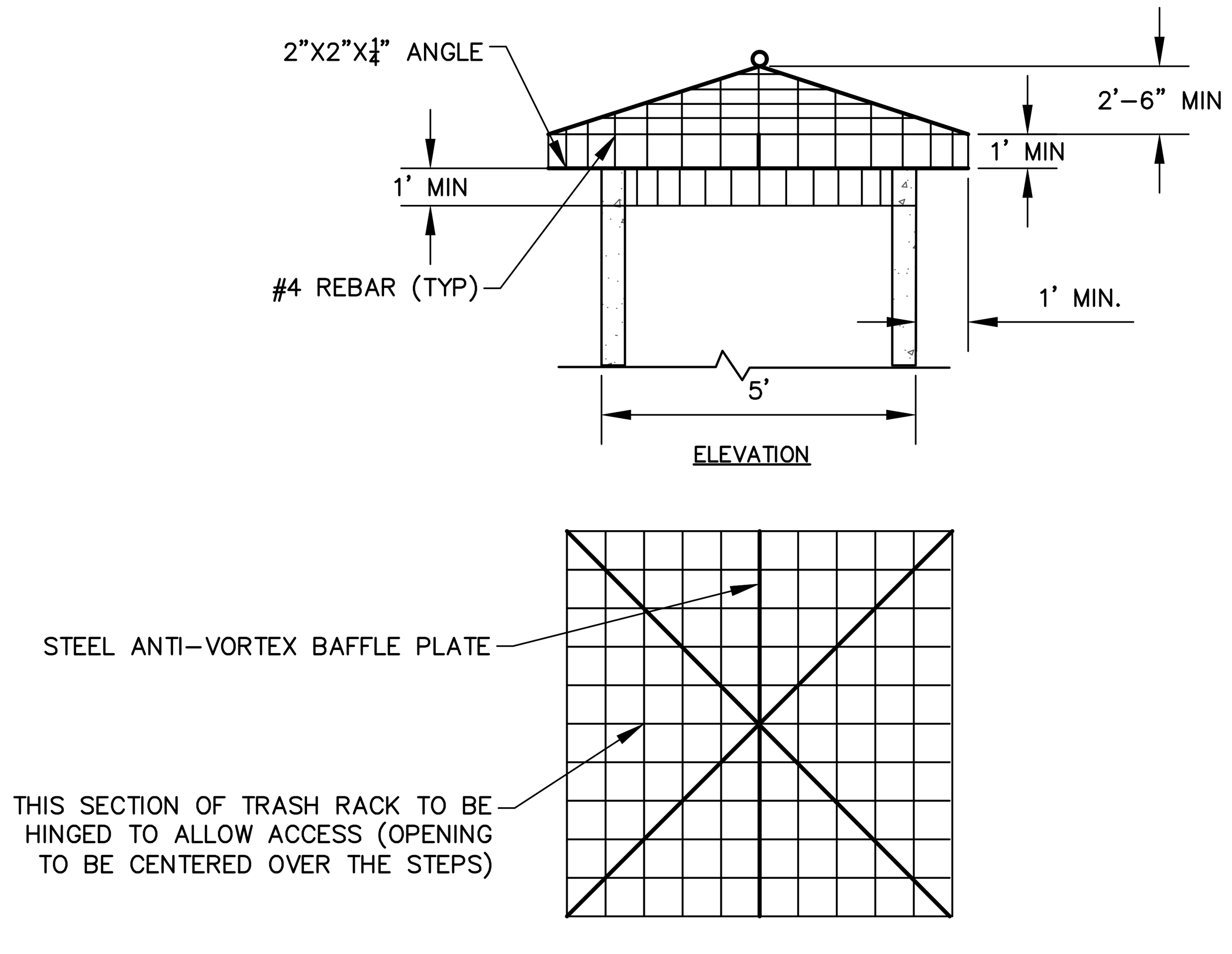
B 23 23 **FRONT VIEW**
NOT TO SCALE



5 14 15 23 **EMERGENCY SPILLWAY**
NOT TO SCALE

NOTES:

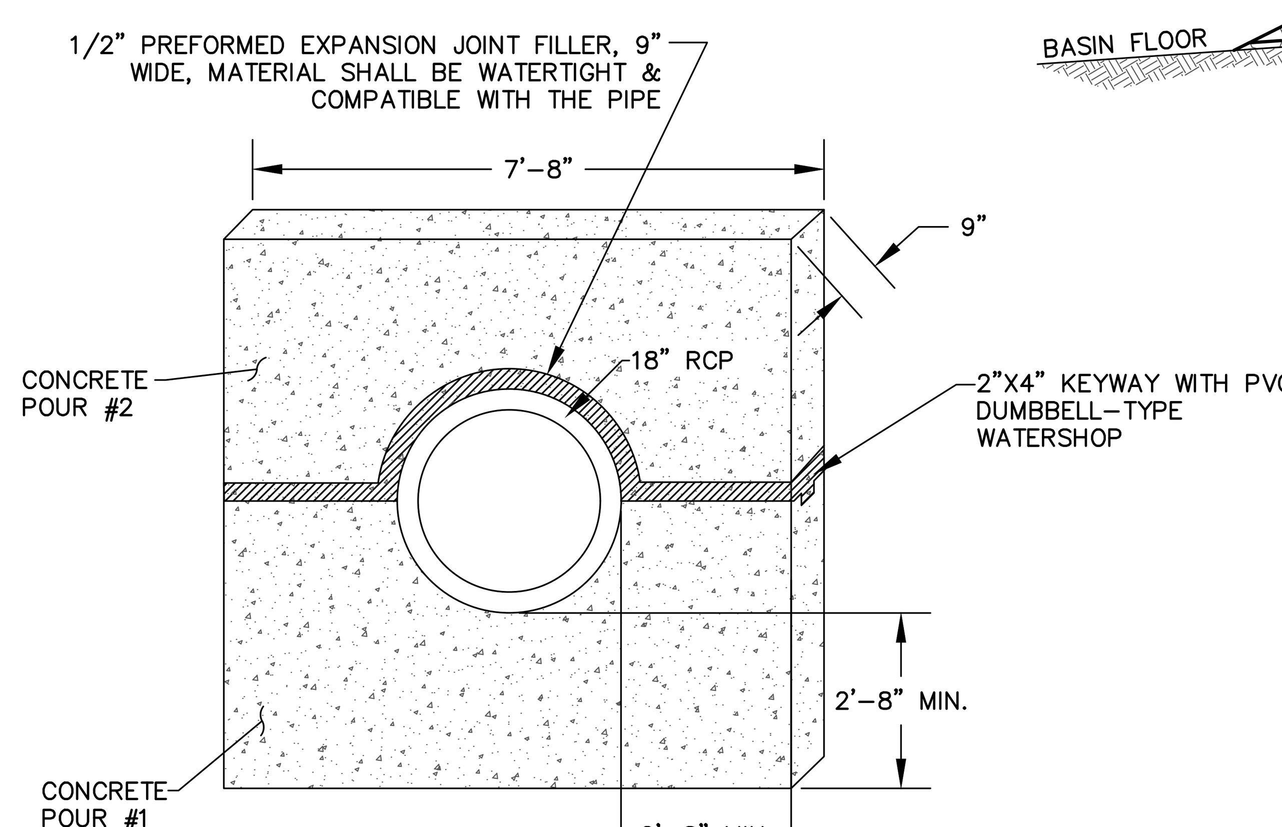
1. EMERGENCY SPILLWAY SURFACE LINING SHALL BE TRM WITH VEGETATIVE GRASS COVER ALONG THE FILL EMBANKMENT.
2. TURF REINFORCEMENT MATTING (TRM) SHALL BE GREENARMOR 7010 OR APPROVED EQUIVALENT.



2 23 23 **TRASH RACK**
NOT TO SCALE

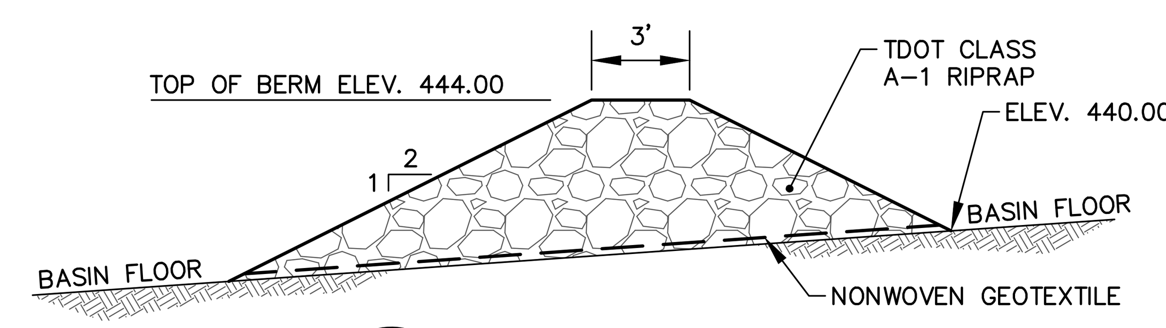
NOTES:

1. ALL REBAR AND ANGLES TO BE GALVANIZED WITH A BITUMINOUS COATING.
2. GAPS TO BE NO SMALLER THAN 6"X6" OR LARGER THAN 12"X12".
3. ANCHOR TRASH RACK TO RISER WALL WITH FOUR EQUALLY SPACED HOT-DIPPED GALVANIZED STEEL CLAMPS. EACH CLAMP ATTACHED TO MANHOLE RISER BY CONCRETE ANCHOR BOLTS.

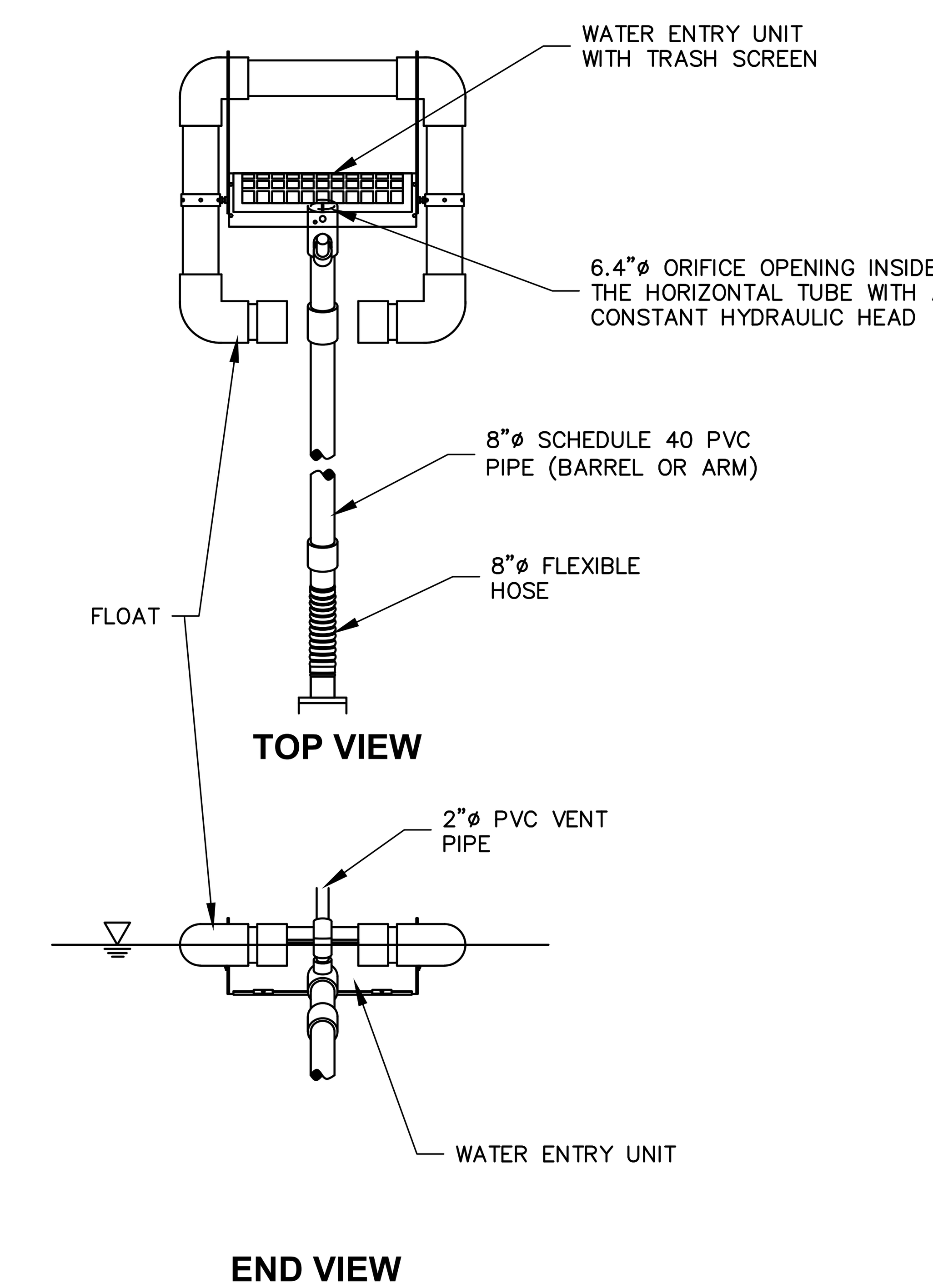
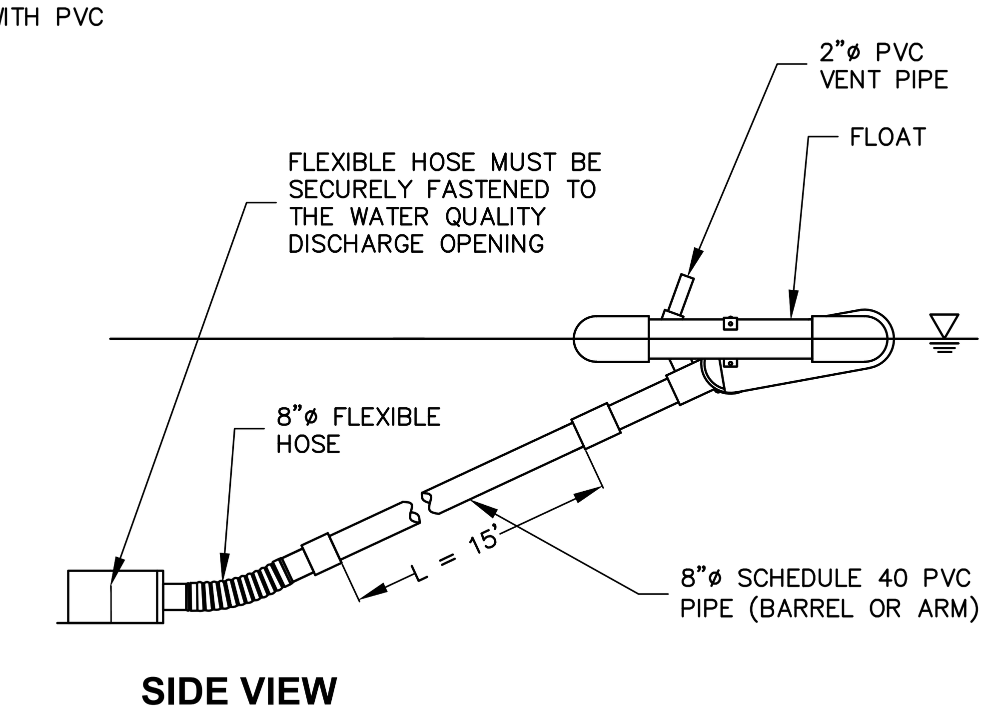


NOTE:
1. CONCRETE SHALL BE 3,000 PSI AT 28 DAYS.
2. ANTI-SEEP COLLAR SHALL BE PLACED AT LEAST 2 FEET FROM NEAREST JOINT.
3. ANIT-SEEP COLLAR SHALL BE CONSTRUCTED IN TWO SECTIONS. BOTTOM HALF SHALL BE POURED FIRST, THE PIPE PLACED, THEN TOP POUR.
4. #4 REBAR SHALL BE PLACED ON 12" CENTERS EACH WAY.

3 9 9 **ANTI-SEEP COLLAR**
NOT TO SCALE



6 14 15 23 **FOREBAY BERM**
NOT TO SCALE

4 23 23 **8" FAIRCLOTH SKIMMER DEWATERING DEVICE**
NOT TO SCALE

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NO.	DATE	BY

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4	TDEC PERMIT REVIEW COMMENTS	9/20/17	NSP
5	TDEC PERMIT REVIEW COMMENTS	4/5/18	NSP

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SEDIMENT BASIN DETAILS

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

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PART F
DESIGN CALCULATIONS

APPENDIX F.1

Storm Water Calculations

F.1A Sediment Basin Design (Rev 3)

F.1B Culvert Design (Rev 3)

F.1C Perimeter Channel Design (Rev 3)

F.1D Letdown Design (Rev 3)

F.1E Storm Water Terrace Design (Rev 3)

F.1F Storm Water Terrace Spacing (Rev 2)

F.1G Outlet Protection Design (Rev 3)

F.1.H Exterior Daylight Channel Design (Rev 3)

APPENDIX F.1A

Sediment Basin Design (Rev 3)

AECOM

I. PURPOSE

The purpose of this analysis is to design the sediment basin for the Scepter Class II Disposal Facility Expansion in Waverly, TN consistent with state regulations.

II. SITE AND PROJECT DESCRIPTION

The sediment basin design was performed as part of the Part II Solid Waste Permit Application. The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management. Refer to Exhibit 1 for a depiction of the sediment basin.

The following sections summarize the design criteria, procedure, assumptions, and results of the sediment basin design.

III. REGULATORY REQUIREMENTS / DESIGN CRITERIA

The below parts of the TDEC regulations specify requirements for the design of the sediment basins:

Rule 0400-11-01-.04(2)(i)2.

The operator must design, construct, operate, and maintain a run-off management system to collect and control at least the peak flow resulting from a 24-hour, 25 year storm.

Rule 0400-11-01-.04(2)(i)3.

Holding facilities (e.g. sediment basins) associated with run-on and run-off control systems must be designed to detain at least the water volume resulting from a 24-hour, 25 year storm and to divert through emergency spillways at least the peak flow resulting from a 24-hour, 100 year storm.

The results of the analysis presented herein show that the sediment basin is designed to detain at least the water volume resulting from a 24-hour, 25 year storm and control discharge flow resulting from a 100-year, 24-hour storm through the emergency spillway.

Rule 0400-11-01-.04(2)(i)4.

Collection and holding facilities associated with run-on and run-off control systems must be emptied or otherwise managed expeditiously after storms to maintain design capacity of the system.

The sediment basin is designed with a low-flow dewatering system to gradually lower the water surface elevation back to the design level following rain events. Total drawdown occurs in 72 hours.

IV. PROCEDURE

Design of the landfill site stormwater features was an iterative process beginning with basic assumptions and a proposed grading plan for the site. The hydraulic features of the sediment basins were initially assumed and then confirmed through multiple iterations.

The AutoCAD Civil 3D software package was used to generate the proposed site grading plan and subsequently to determine drainage areas, volumes, and other site geometry. HydroCAD (version 10.00) modeling software was used to conduct the hydrologic and hydraulic calculations for this analysis with inputs based on the site geometry, rainfall data, and other design assumptions.

The model was used to generate surface water volumes and peak water surface elevations in the basin for the required design storm conditions and based on upstream watershed features of site and the spillway design parameters.

The sediment basin was sized such that the volume provided in the basin between the bottom of the basin's dry storage/top of the basin's wet pool and the emergency spillway overflow elevation is greater than the total predicted volume of inflow entering the basin during the 24-hour, 25-year storm event. By providing this volume, the basin is capable of detaining at least the water volume resulting from a 24-hour, 25 year storm, in accordance with TDEC Rule 0400-11-01-.04(2)(i)3.

The proposed primary spillway riser will pass all discharge flows equal to and lesser than those generated by the 24-hour, 100-year storm, while providing a minimum 1-foot of freeboard from overtopping the surrounding embankment. Additionally, an earthen emergency spillway provides redundancy to the primary spillway riser and will pass all discharge flows equal to and lesser than those generated by the 24-hour, 100-year storm (in accordance with TDEC Rule 0400-11-01-.04(2)(i)3), while providing a minimum 1-foot of freeboard from overtopping the surrounding embankment.

A low-flow dewatering system was designed to lower the water elevation in the pond from the invert of the proposed spillway to the dewatering (wet storage) elevation over the course of 72 hours. The dewatering elevation was set such that the wet and dry storage available in the pond exceeds the required minimum storage volumes. The required minimum volume for wet storage is based on 67 cubic yards per acre of contributing drainage area per the TDEC Erosion and Sediment Control Handbook Section 7.31 Sediment Basin. The dry storage requirement is also 67 cubic yards per acre per the TDEC Erosion and Sediment Control Handbook, but is exceeded by Rule 0400-11-01-.04(2)(i)2 of the TDEC Solid Waste Management regulations and the dry storage is equivalent to the 24-hour, 25-year storm volume. The low-flow dewatering system is a combination of orifices and a surface skimmer. The surface skimmer will drain the final 67 cubic yards per acre of dry storage volume independently from the orifices. In other words, the orifices are set above the TDEC Erosion and Sediment Control Handbook requirements for wet storage allowing the surface skimmer to dewater the final 67 cubic yards per acre volume.

As part of the dry storage volume, the sediment basin includes a forebay designed in accordance with TDEC Erosion and Sediment Control Handbook Section 7.31 Sediment Basin. The forebay volume exceeds the 16.75 cubic yards per acre (25% of the 67 cubic yards per acre dry/wet pool requirement) minimum requirement set forth by the TDEC Erosion and Sediment Control Handbook. The forebay is separated from the wet storage by a porous (TDOT Class A-1 riprap) berm.

V. NOTES/ASSUMPTIONS

The following is a list of key notes and assumptions made in completing this analysis.

- The rainfall intensities for Waverly, Tennessee are as follows and were obtained from NOAA.gov and are included as Attachment A.:
  - 6.39 inches to represent the 24-hour, 25-year storm
  - 8.12 inches to represent the 24-hour, 100-year storm
- Within the HydroCAD program, the runoff was calculated using the SCS TR-20 method.
- Runoff curve numbers (CN) used in the analysis were as follows:
  - 78 for landfill vegetated final cap cover and offsite areas,
  - 98 for sediment basin water surface.
  - 77 for exposed and disturbed soil (daily cover); however exposed and disturbed soil was ignored to be conservative and 78 was used within the landfill construction footprint.
- The time of concentration was calculated using the Curve Number Method in HydroCAD which takes inputs for each drainage area of the longest hydraulic flow path and average land slope.
- For the purposes of designing the spillway structure, the dewatering devices in the basins were not included in the hydraulic model.
- For the purposes of designing the spillway structure, the starting water surface elevation (WSE) in the basins was assumed to be at the elevation of the wet pool.
- Pipe flow in HydroCAD was calculated by Manning's equation for low flow conditions and the orifice equation for high flow (submerged) conditions.

- Flow into the spillway riser in HydroCAD was calculated using weir flow for low flow conditions and the orifice equation for high flow (submerged) conditions.

VI. SUMMARY OF RESULTS

The results of the HydroCAD modeling for the sediment basin are provided in Attachment B (25-year, 24-hour storm event), and Attachment C (100-year, 24-hour storm event). The 72-hour dry pool dewatering can be found in Attachment D.

The results of the sediment basin design are summarized in Table 1. The basin both detains the water volume generated by the 24-hour, 25-year storm and the discharge structure is sized appropriately such that the peak water surface elevation in the basin during the 24-hour, 100-year storm stays safely below the peak storage elevation of the basin meeting the one foot freeboard requirement. The proposed spillway structures consist of 48” x 48” box riser grated inlets discharging via a single 18” diameter reinforced concrete pipe (RCP). A beehive type grate will be placed over the box riser to protect against clogging and trapping debris.

The earthen emergency spillway surface lining is compacted native soil fill within the influence of the access road and vegetated grass outside the influence of the access road. Refer to Attachment F for detailed calculations demonstrating the maximum shear stress (0.066 lb/ft2) from the emergency spillway along the access road is less than the permissible shear stress of compacted native soil (0.075 lb/ft^2). Along the back slope of the emergency spillway the maximum design velocity is 3.7 feet per second (Attachment F). 3.7 feet per second is under the maximum 5.0 feet per second threshold for grass lined spillways confirming a vegetated grass lined spillway is adequate.

Table 1 – Sediment Basin Hydraulic Design Summary

	Fully Operational System	Without Low-Flow Dewatering System[1] (For Primary Spillway Riser Overflow Design)	Without Low-Flow Dewatering and Primary Spillway Systems[1] (For Emergency Spillway Design)
Total Contributing Drainage Area (acres)	31.05		
Faircloth Skimmer Invert Elevation (feet)	439.25	Not Included[1]	Not Included[1]
Five 1.50" Dia. Orifices	441.25	Not Included[1]	Not Included[1]
Five 1.50" Dia. Orifices	442.25	Not Included[1]	Not Included[1]
Five 1.50" Dia. Orifices	443.25	Not Included[1]	Not Included[1]
Five 1.50" Dia. Orifices	444.25	Not Included[1]	Not Included[1]
Five 1.50" Dia. Orifices	445.25	Not Included[1]	Not Included[1]
Five 1.50" Dia. Orifices	446.25	Not Included[1]	Not Included[1]
Five 1.50" Dia. Orifices	447.25	Not Included[1]	Not Included[1]
Five 1.50" Dia. Orifices	448.25	Not Included[1]	Not Included[1]
Primary Spillway Riser Overflow Elevation (feet)	450.00		Not Included[1]
Primary Spillway Pipe Inlet Invert Elevation (feet)	438.75		Not Included[1]
Primary Spillway Pipe Outlet Invert Elevation (feet)	434.38		Not Included[1]
Earthen Emergency Spillway Invert (feet)	450.25		
Basin Crest Elevation (feet)	451.50		
Peak Storage at Basin Crest Elevation (acre-feet)[2]	14.69		
24-hour, 25-year storm - 6.39 inches			
Surface Water Runoff Volume Generated (acre-feet)	10.43		
Storage Volume to Spillway Overflow Elevation (acre-feet)[3]	10.91		Not Applicable
Peak Water Elevation in Basin (feet)	447.32	449.67	
Peak Discharge – Primary Spillway (cfs)	3.97	0.00	Not Applicable
Peak Discharge – Emergency Spillway (cfs)	0.00		
24-hour, 100-year storm – 8.12 inches			
Peak Water Elevation in Basin (feet)	449.44	450.25	450.45
Peak Storage[3] (acre-feet)	10.09	11.29	11.59
Peak Discharge – Principal Spillway (cfs)	5.64	7.08	0.00
Peak Discharge – Emergency Spillway (cfs)	0.00		5.84

[1] Assume system failure
[2] Including wet storage volume (1.43 acre-feet)
[3] Excluding wet storage volume (1.43 acre-feet)

The results of the low-flow dewatering device and sediment treatment design are summarized in Table 2 below.

Table 2 - Sediment Basin Treatment Design Summary

	Basin 1
Bottom of Pond Elevation (ft)	434.00
Dewatering Device Elevation (ft)	439.25
Wet Storage Available (acre-feet)	1.30
Wet Storage Required (acre-feet)	1.29
Dry Storage Available (acre-feet)	10.91
Dry Storage Required (acre-feet)	10.43
Forebay Storage Available (acre-feet)	0.50
Forebay Storage Required (acre-feet)	0.33

Based upon information provided on fairclothskimmer.com, a 8" Faircloth Skimmer along with 40 – 1.50" diameter orifices are capable of drawing down the designed dry storage available within the required 72 hour time frame (see Attachments E for Faircloth Skimmer calculations; note a 6.4" diameter orifice will be installed to achieve the required drawdown rate of the 8" Faircloth Skimmer). If a different surface skimmer product is selected for installation, calculations will be needed to confirm an appropriate drawdown rate can be achieved to meet requirements discussed herein.

VII. CONCLUSIONS

The proposed grading of the landfill and sediment basin in combination with the design of the primary and emergency spillway structures are sufficient to safely control and convey the design storms as stipulated by TDEC Solid Waste Management regulations. A low-flow dewatering system will gradually lower the pond water surface elevation after storm events to regain maximum storage capacity while allowing for trapping of sediment within the wet storage zone at the pond bottom. Calculations for proper conveyance of upstream and downstream flows were not presented in this analysis and will be performed separately. Refer to accompanying calculations used to design upstream conveyance features (storm water terraces, perimeter channels, and culverts) entering the basins.

VIII. ATTACHMENTS

Figures:

Exhibit 1: Drainage Map

Attachments:

Attachment A: NOAA Precipitation Frequency Data
Attachment B: HydroCAD report for Sediment Basin - 24-hour, 25-year storm
Attachment C: HydroCAD report for Sediment Basin - 24-hour, 100-year storm
Attachment D: HydroCAD report for Sediment Basin - 72-hour dry pool dewatering
Attachment E: Faircloth Skimmer Sizing Calculator
Attachment F: Emergency Spillway Lining

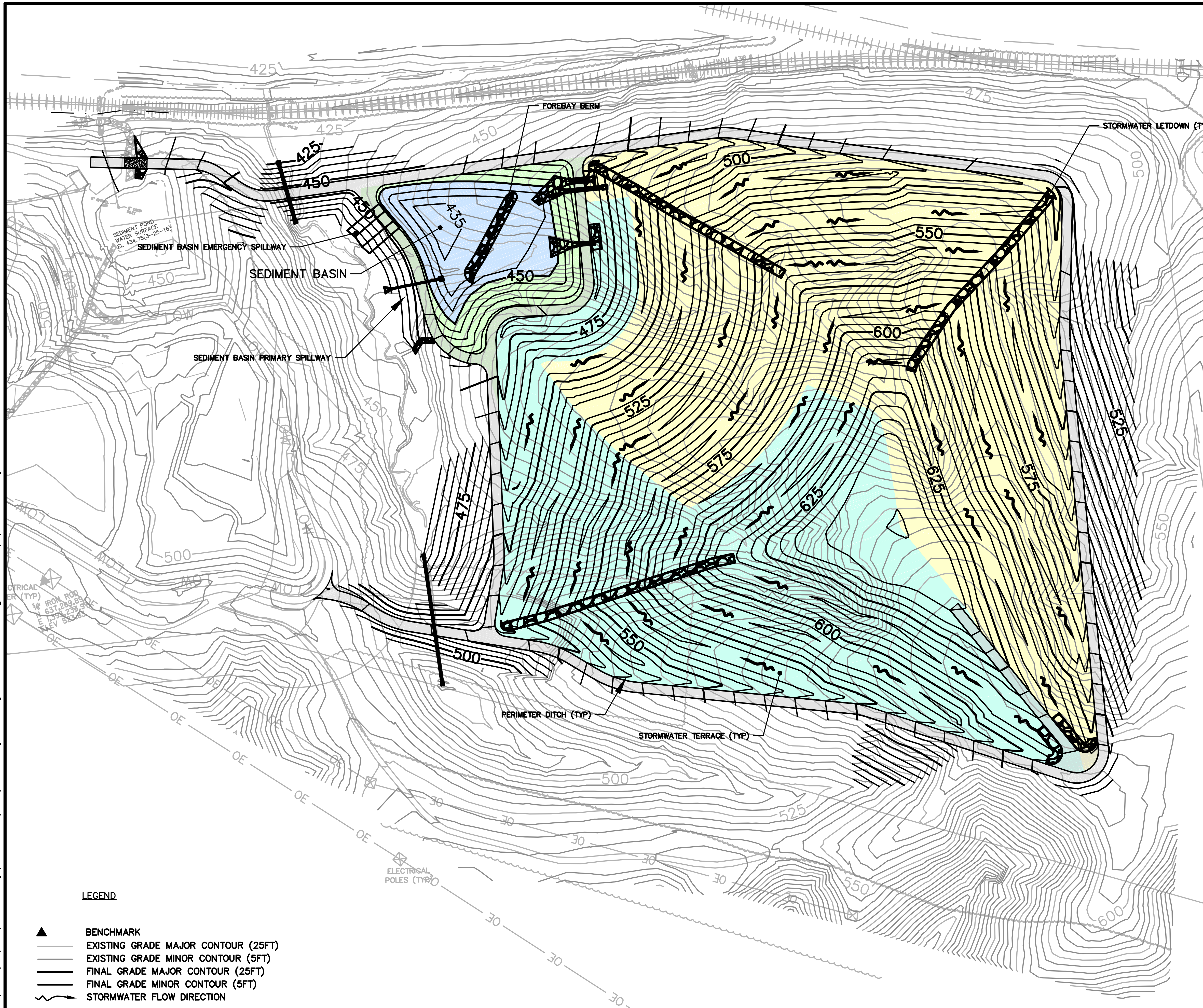
IX. REFERENCES

1. TDEC, *"Rules of Tennessee Department of Environment and Conservation, Chapter 0400-11-01 – Solid Waste Processing and Disposal"*, Solid Waste Management, September 2012.

2. TDEC, *"Erosion and Sediment Control Handbook"*, Fourth Edition, August 2012.

EXHIBIT 1

Drainage Map



S:\2016\Scepter\028 Waverly\02 Landfill\CAD\x-tributary boundary-sediment basin.dwg User: nick.popkowski May 04, 2017 - 9:32am

LEGEND

- ▲ BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (25FT)
- EXISTING GRADE MINOR CONTOUR (5FT)
- FINAL GRADE MAJOR CONTOUR (25FT)
- FINAL GRADE MINOR CONTOUR (5FT)
- STORMWATER FLOW DIRECTION

DRAINAGE AREAS

NORTH SED. BASIN CULVERT AREA = 16.28 ACRES CN = 78 TC = 14.0 MINS
SOUTH SED. BASIN CULVERT AREA = 12.36 ACRES CN = 78 TC = 16.5 MINS
SED. BASIN OPEN WATER AREA = 1.50 ACRES CN = 98 TC = 5.0 MINS
SED. BASIN SOIL AREA = 1.50 ACRES CN = 78 TC = 5.0 MINS

AECOM

SCEPTER, INC.

WAVERLY, TENNESSEE

SEDIMENT BASIN DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/4/17	1

ATTACHMENT A

NOAA Precipitation Frequency Data

NOAA Atlas 14, Volume 2, Version 3
Location name: Waverly, Tennessee, US*
Latitude: 36.0689°, Longitude: -87.9462°
Elevation: 523 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.405 (0.368-0.450)	0.476 (0.434-0.529)	0.547 (0.497-0.607)	0.604 (0.548-0.668)	0.675 (0.610-0.746)	0.728 (0.655-0.803)	0.781 (0.699-0.862)	0.832 (0.742-0.917)	0.898 (0.794-0.990)	0.948 (0.833-1.05)
10-min	0.647 (0.588-0.718)	0.761 (0.694-0.845)	0.876 (0.796-0.972)	0.965 (0.877-1.07)	1.08 (0.972-1.19)	1.16 (1.04-1.28)	1.24 (1.11-1.37)	1.32 (1.18-1.45)	1.42 (1.26-1.57)	1.49 (1.31-1.65)
15-min	0.809 (0.735-0.898)	0.957 (0.872-1.06)	1.11 (1.01-1.23)	1.22 (1.11-1.35)	1.36 (1.23-1.51)	1.47 (1.32-1.62)	1.57 (1.40-1.73)	1.66 (1.48-1.83)	1.79 (1.58-1.97)	1.88 (1.65-2.07)
30-min	1.11 (1.01-1.23)	1.32 (1.20-1.47)	1.58 (1.43-1.75)	1.77 (1.61-1.96)	2.02 (1.83-2.23)	2.21 (1.99-2.44)	2.40 (2.15-2.65)	2.59 (2.31-2.86)	2.84 (2.51-3.14)	3.04 (2.67-3.35)
60-min	1.38 (1.26-1.54)	1.66 (1.51-1.84)	2.02 (1.83-2.24)	2.30 (2.09-2.55)	2.69 (2.43-2.97)	2.99 (2.70-3.31)	3.31 (2.96-3.65)	3.63 (3.24-4.01)	4.08 (3.61-4.50)	4.43 (3.89-4.89)
2-hr	1.59 (1.44-1.76)	1.91 (1.73-2.11)	2.33 (2.11-2.59)	2.69 (2.43-2.98)	3.19 (2.87-3.52)	3.60 (3.22-3.98)	4.03 (3.59-4.46)	4.49 (3.98-4.97)	5.14 (4.52-5.70)	5.67 (4.95-6.29)
3-hr	1.72 (1.57-1.89)	2.05 (1.87-2.27)	2.52 (2.29-2.78)	2.91 (2.64-3.20)	3.45 (3.12-3.80)	3.91 (3.52-4.30)	4.39 (3.93-4.82)	4.90 (4.37-5.39)	5.64 (4.98-6.20)	6.24 (5.47-6.88)
6-hr	2.13 (1.93-2.35)	2.53 (2.30-2.80)	3.10 (2.82-3.43)	3.58 (3.25-3.95)	4.26 (3.84-4.69)	4.83 (4.33-5.32)	5.43 (4.85-5.98)	6.09 (5.40-6.70)	7.01 (6.17-7.72)	7.78 (6.78-8.58)
12-hr	2.58 (2.36-2.85)	3.08 (2.82-3.40)	3.79 (3.46-4.18)	4.38 (3.99-4.82)	5.22 (4.73-5.74)	5.92 (5.34-6.51)	6.66 (5.97-7.32)	7.46 (6.65-8.20)	8.60 (7.59-9.45)	9.54 (8.36-10.5)
24-hr	3.14 (2.88-3.44)	3.76 (3.45-4.12)	4.64 (4.25-5.09)	5.37 (4.91-5.87)	6.39 (5.82-6.98)	7.23 (6.56-7.89)	8.12 (7.32-8.85)	9.06 (8.12-9.88)	10.4 (9.22-11.3)	11.5 (10.1-12.5)
2-day	3.75 (3.46-4.10)	4.49 (4.14-4.92)	5.53 (5.09-6.04)	6.36 (5.84-6.93)	7.52 (6.89-8.19)	8.46 (7.72-9.21)	9.45 (8.56-10.3)	10.5 (9.44-11.4)	11.9 (10.6-13.0)	13.1 (11.6-14.3)
3-day	4.01 (3.69-4.37)	4.80 (4.42-5.24)	5.89 (5.43-6.43)	6.77 (6.22-7.37)	7.98 (7.31-8.69)	8.96 (8.17-9.75)	9.97 (9.04-10.9)	11.0 (9.94-12.0)	12.5 (11.2-13.7)	13.6 (12.1-15.0)
4-day	4.26 (3.93-4.64)	5.11 (4.71-5.56)	6.26 (5.77-6.82)	7.18 (6.61-7.81)	8.45 (7.74-9.18)	9.46 (8.62-10.3)	10.5 (9.52-11.4)	11.6 (10.4-12.6)	13.0 (11.7-14.3)	14.2 (12.6-15.6)
7-day	5.06 (4.65-5.50)	6.06 (5.58-6.59)	7.43 (6.83-8.07)	8.52 (7.81-9.25)	10.0 (9.17-10.9)	11.3 (10.2-12.2)	12.6 (11.4-13.6)	13.9 (12.5-15.1)	15.7 (14.0-17.2)	17.2 (15.2-18.8)
10-day	5.77 (5.34-6.22)	6.90 (6.38-7.45)	8.39 (7.75-9.05)	9.55 (8.81-10.3)	11.1 (10.2-12.0)	12.4 (11.3-13.3)	13.6 (12.4-14.7)	14.9 (13.5-16.1)	16.6 (15.0-18.0)	18.0 (16.1-19.6)
20-day	7.88 (7.35-8.46)	9.36 (8.74-10.1)	11.1 (10.4-12.0)	12.5 (11.6-13.4)	14.2 (13.2-15.3)	15.5 (14.4-16.7)	16.8 (15.5-18.1)	18.1 (16.6-19.5)	19.7 (18.0-21.2)	20.9 (19.0-22.6)
30-day	9.63 (9.00-10.3)	11.4 (10.7-12.2)	13.4 (12.6-14.4)	15.0 (14.0-16.0)	16.9 (15.8-18.1)	18.4 (17.1-19.7)	19.9 (18.5-21.3)	21.4 (19.7-22.9)	23.2 (21.3-24.9)	24.6 (22.5-26.4)
45-day	12.1 (11.3-12.9)	14.3 (13.4-15.2)	16.7 (15.7-17.8)	18.5 (17.3-19.7)	20.8 (19.4-22.1)	22.4 (20.9-23.9)	24.1 (22.4-25.6)	25.6 (23.7-27.3)	27.6 (25.4-29.5)	29.0 (26.6-31.0)
60-day	14.5 (13.6-15.4)	17.1 (16.0-18.2)	19.8 (18.6-21.1)	21.8 (20.4-23.2)	24.3 (22.7-25.9)	26.1 (24.4-27.9)	27.8 (25.9-29.7)	29.5 (27.4-31.5)	31.4 (29.1-33.6)	32.9 (30.3-35.2)

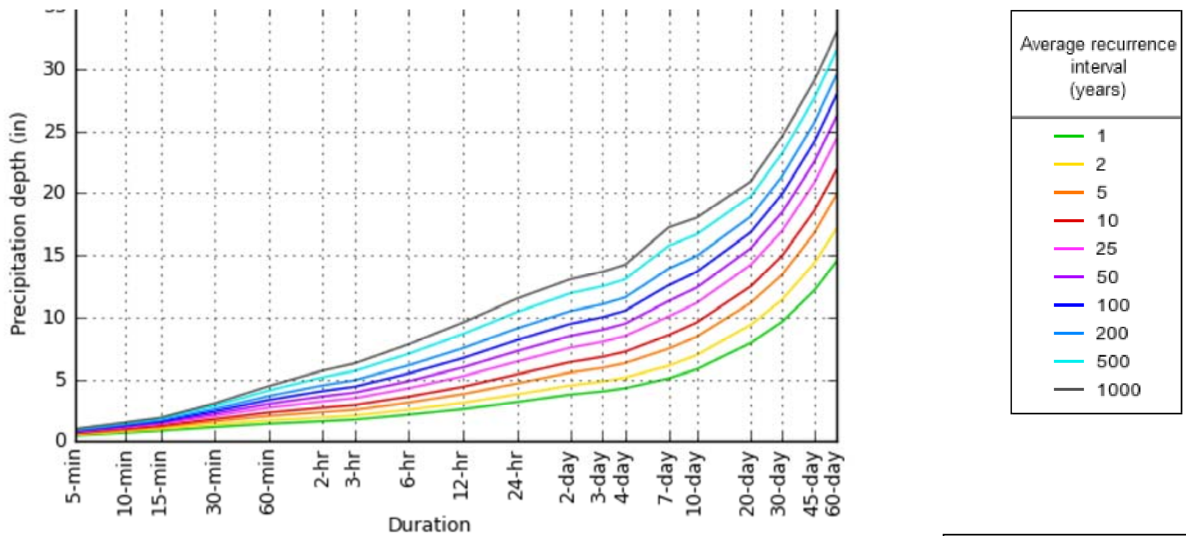
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

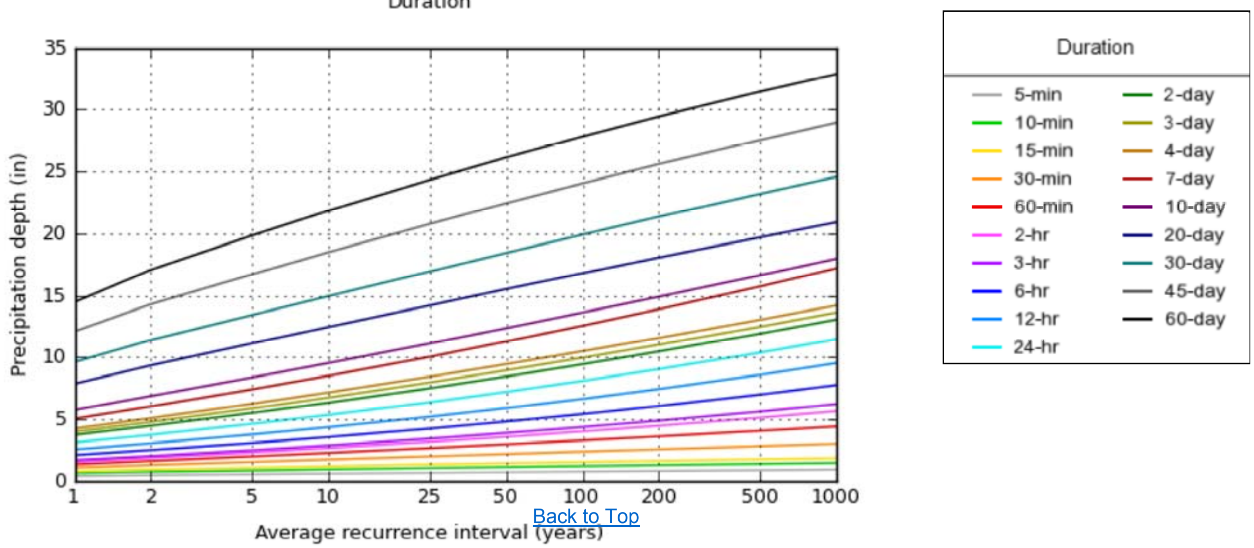
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical





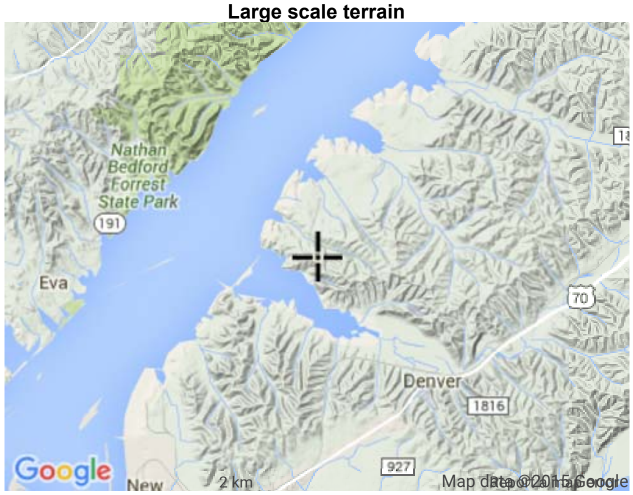
[Back to Top](#)

NOAA Atlas 14, Volume 2, Version 3 Created (GMT): Tue Dec 1 22:54:15 2015

Maps & aerials

Small scale terrain

Large scale terrain



Large scale map

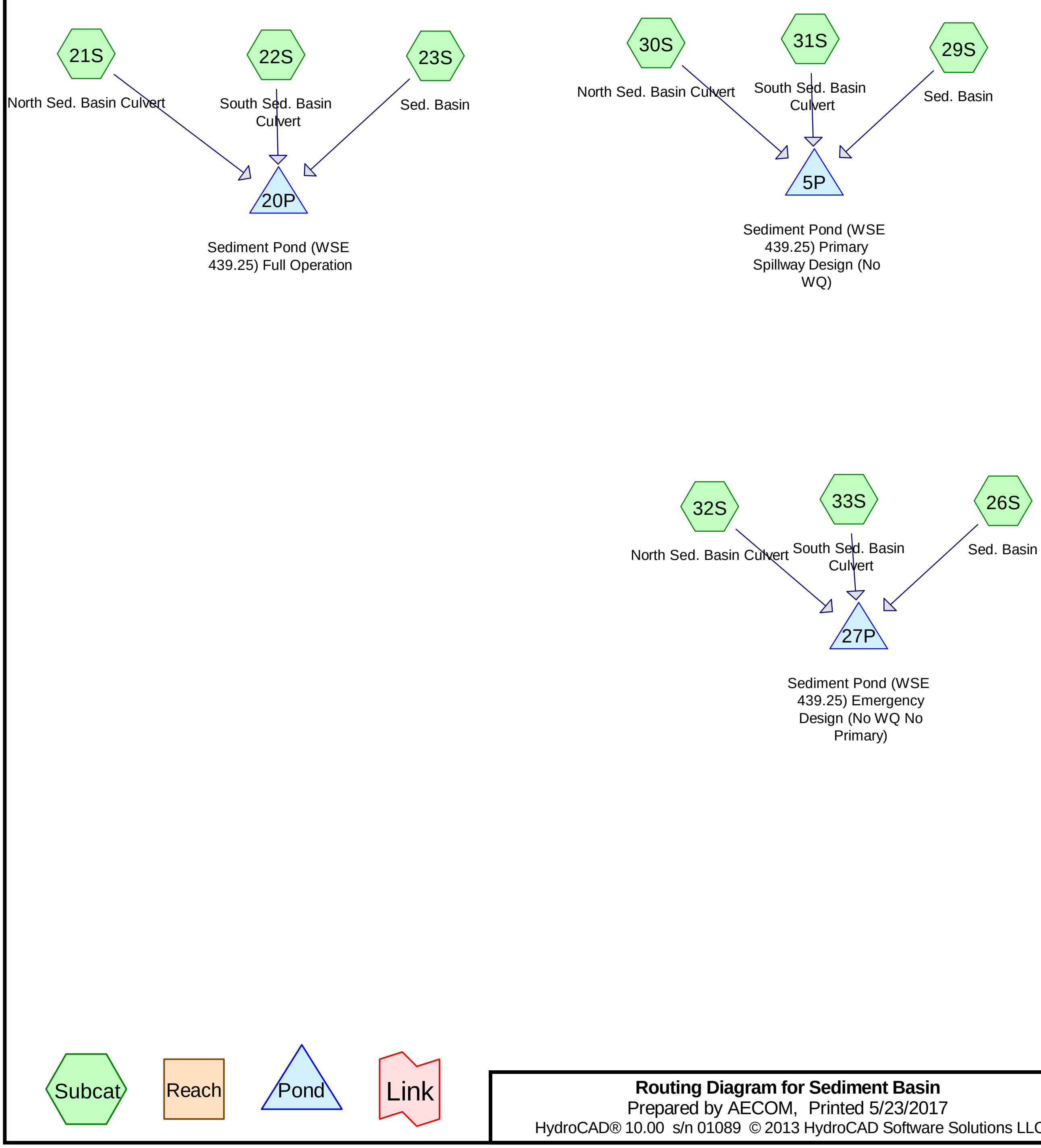
Large scale aerial

[Back to Top](#)

[US Department of Commerce](#)
[National Oceanic and Atmospheric Administration](#)
[National Weather Service](#)
[National Water Center](#)
1325 East West Highway
Silver Spring, MD 20910

ATTACHMENT B

HydroCAD report for Sediment Basin - 24-hr, 25-yr storm



Routing Diagram for Sediment Basin
Prepared by AECOM, Printed 5/23/2017
HydroCAD® 10.00 s/n 01089 © 2013 HydroCAD Software Solutions LLC

Summary for Subcatchment 21S: North Sed. Basin Culvert

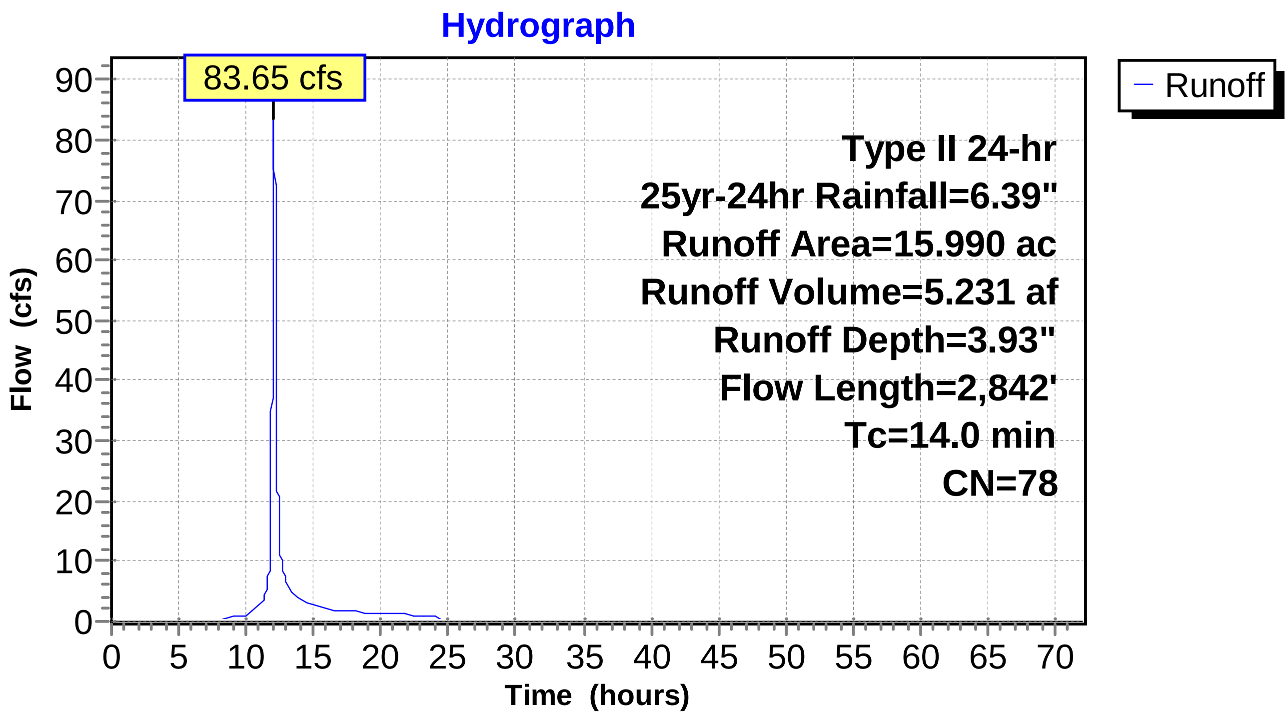
Runoff = 83.65 cfs @ 12.06 hrs, Volume= 5.231 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow**
					Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow**
					Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace**
					Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down**
					Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1**
					Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2**
					Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3**
					Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4**
					Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 21S: North Sed. Basin Culvert



Hydrograph for Subcatchment 21S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.06
8.00	0.77	0.01	0.25
9.00	0.94	0.04	0.62
10.00	1.16	0.10	1.12
11.00	1.50	0.23	2.67
12.00	4.24	2.08	**73.26**
13.00	4.93	2.65	**6.29**
14.00	5.24	2.92	3.64
15.00	5.45	3.10	2.81
16.00	5.62	3.25	2.20
17.00	5.76	3.37	1.90
18.00	5.89	3.48	1.68
19.00	5.99	3.57	1.46
20.00	6.08	3.65	1.23
21.00	6.16	3.72	1.15
22.00	6.24	3.79	1.11
23.00	6.32	3.86	1.07
24.00	**6.39**	**3.93**	1.02
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 22S: South Sed. Basin Culvert

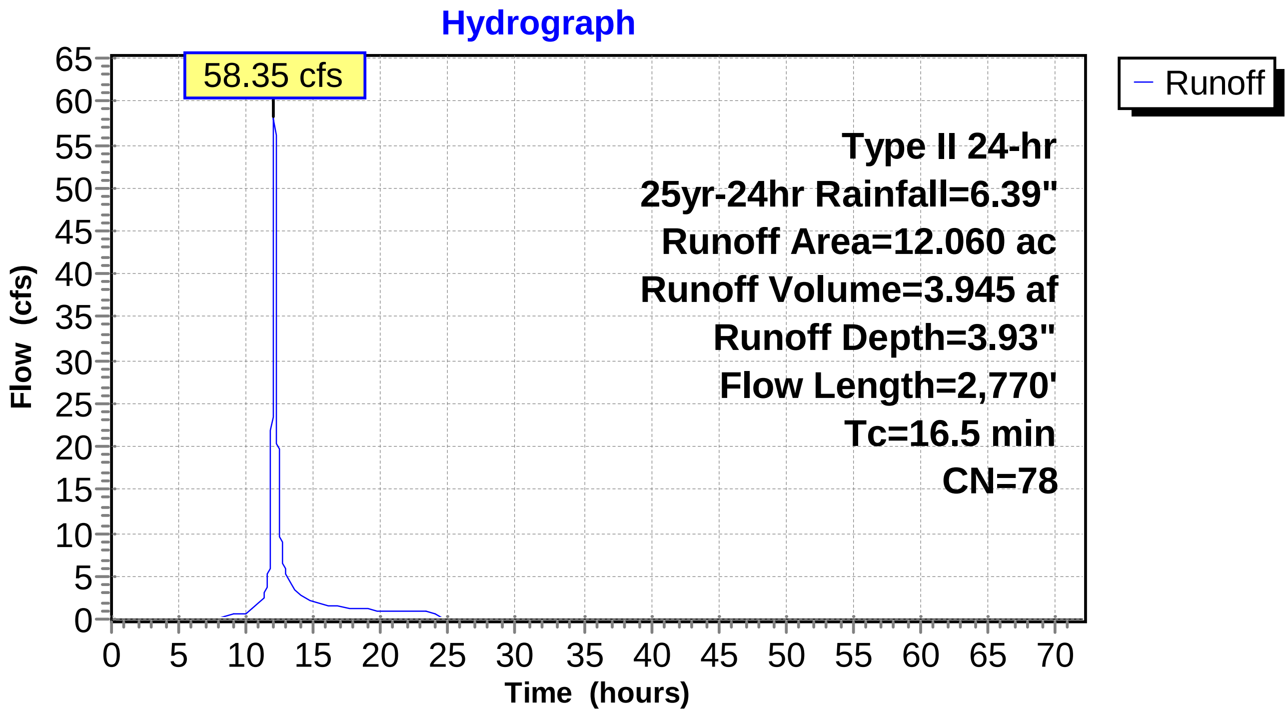
Runoff = 58.35 cfs @ 12.08 hrs, Volume= 3.945 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 22S: South Sed. Basin Culvert



Hydrograph for Subcatchment 22S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.04
8.00	0.77	0.01	0.18
9.00	0.94	0.04	0.45
10.00	1.16	0.10	0.82
11.00	1.50	0.23	1.94
12.00	4.24	2.08	**46.36**
13.00	4.93	2.65	**4.91**
14.00	5.24	2.92	2.80
15.00	5.45	3.10	2.13
16.00	5.62	3.25	1.68
17.00	5.76	3.37	1.44
18.00	5.89	3.48	1.27
19.00	5.99	3.57	1.11
20.00	6.08	3.65	0.94
21.00	6.16	3.72	0.87
22.00	6.24	3.79	0.84
23.00	6.32	3.86	0.81
24.00	**6.39**	**3.93**	0.77
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 23S: Sed. Basin

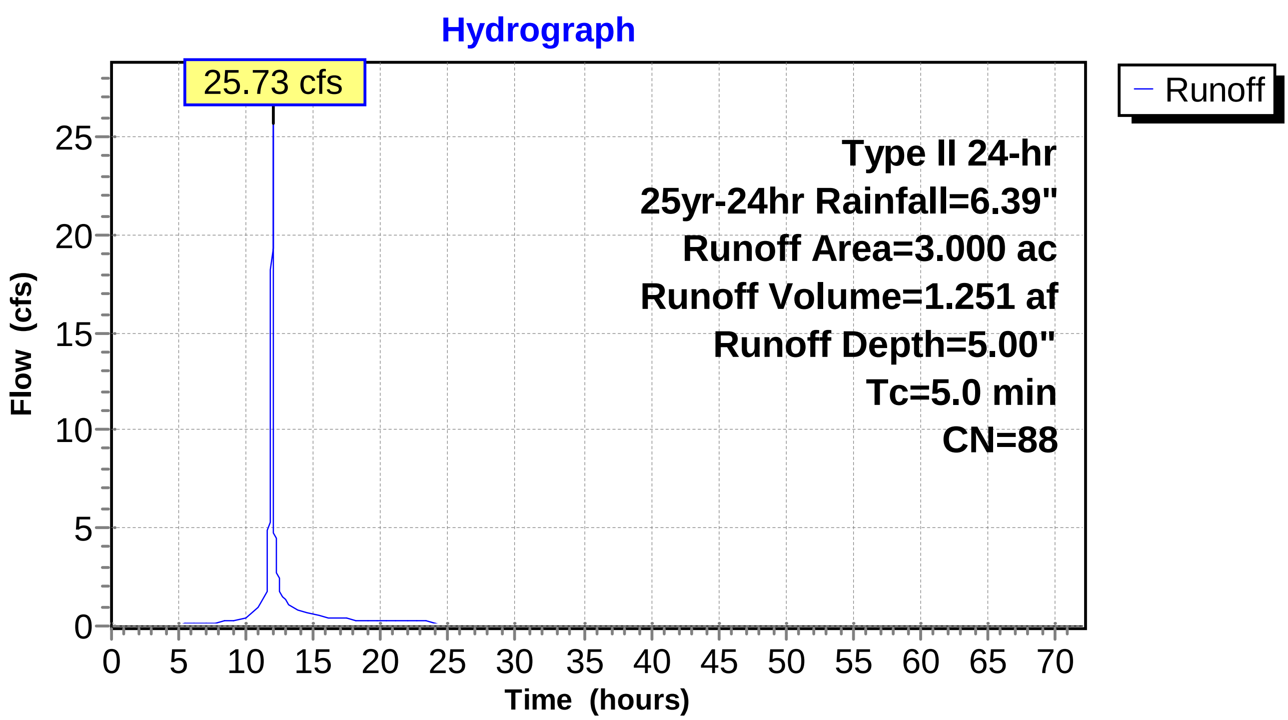
Runoff = 25.73 cfs @ 11.96 hrs, Volume= 1.251 af, Depth= 5.00"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	1.500	98	
*	1.500	78	
	3.000	88	Weighted Average
	1.500		50.00% Pervious Area
	1.500		50.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry, Min. Time of Concentration**

Subcatchment 23S: Sed. Basin



Hydrograph for Subcatchment 23S: Sed. Basin

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)	Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00	54.00	6.39	5.00	0.00
1.00	0.07	0.00	0.00	55.00	6.39	5.00	0.00
2.00	0.14	0.00	0.00	56.00	6.39	5.00	0.00
3.00	0.22	0.00	0.00	57.00	6.39	5.00	0.00
4.00	0.31	0.00	0.01	58.00	6.39	5.00	0.00
5.00	0.40	0.01	0.05	59.00	6.39	5.00	0.00
6.00	0.51	0.04	0.09	60.00	6.39	5.00	0.00
7.00	0.63	0.08	0.14	61.00	6.39	5.00	0.00
8.00	0.77	0.13	0.19	62.00	6.39	5.00	0.00
9.00	0.94	0.22	0.33	63.00	6.39	5.00	0.00
10.00	1.16	0.35	0.47	64.00	6.39	5.00	0.00
11.00	1.50	0.58	**0.95**	65.00	6.39	5.00	0.00
12.00	4.24	2.95	**21.48**	66.00	6.39	5.00	0.00
13.00	4.93	3.61	1.19	67.00	6.39	5.00	0.00
14.00	5.24	3.90	0.71	68.00	6.39	5.00	0.00
15.00	5.45	4.10	0.56	69.00	6.39	5.00	0.00
16.00	5.62	4.26	0.44	70.00	6.39	5.00	0.00
17.00	5.76	4.40	0.38	71.00	6.39	5.00	0.00
18.00	5.89	4.52	0.34	72.00	6.39	5.00	0.00
19.00	5.99	4.62	0.29				
20.00	6.08	4.71	0.25				
21.00	6.16	4.78	0.23				
22.00	6.24	4.86	0.22				
23.00	6.32	4.93	0.22				
24.00	**6.39**	**5.00**	0.21				
25.00	6.39	5.00	0.00				
26.00	6.39	5.00	0.00				
27.00	6.39	5.00	0.00				
28.00	6.39	5.00	0.00				
29.00	6.39	5.00	0.00				
30.00	6.39	5.00	0.00				
31.00	6.39	5.00	0.00				
32.00	6.39	5.00	0.00				
33.00	6.39	5.00	0.00				
34.00	6.39	5.00	0.00				
35.00	6.39	5.00	0.00				
36.00	6.39	5.00	0.00				
37.00	6.39	5.00	0.00				
38.00	6.39	5.00	0.00				
39.00	6.39	5.00	0.00				
40.00	6.39	5.00	0.00				
41.00	6.39	5.00	0.00				
42.00	6.39	5.00	0.00				
43.00	6.39	5.00	0.00				
44.00	6.39	5.00	0.00				
45.00	6.39	5.00	0.00				
46.00	6.39	5.00	0.00				
47.00	6.39	5.00	0.00				
48.00	6.39	5.00	0.00				
49.00	6.39	5.00	0.00				
50.00	6.39	5.00	0.00				
51.00	6.39	5.00	0.00				
52.00	6.39	5.00	0.00				
53.00	6.39	5.00	0.00				

Summary for Subcatchment 26S: Sed. Basin

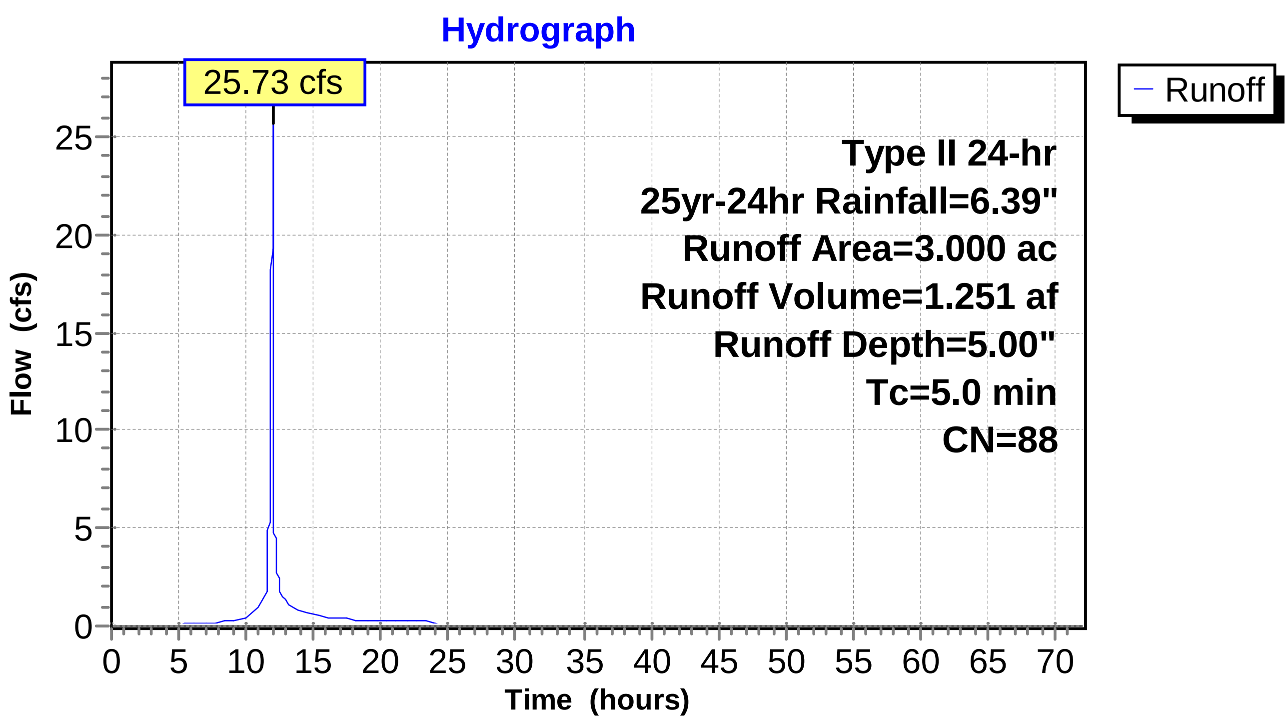
Runoff = 25.73 cfs @ 11.96 hrs, Volume= 1.251 af, Depth= 5.00"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	1.500	98	
*	1.500	78	
	3.000	88	Weighted Average
	1.500		50.00% Pervious Area
	1.500		50.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry, Min. Time of Concentration**

Subcatchment 26S: Sed. Basin



Hydrograph for Subcatchment 26S: Sed. Basin

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.01
5.00	0.40	0.01	0.05
6.00	0.51	0.04	0.09
7.00	0.63	0.08	0.14
8.00	0.77	0.13	0.19
9.00	0.94	0.22	0.33
10.00	1.16	0.35	0.47
11.00	1.50	0.58	**0.95**
12.00	4.24	2.95	**21.48**
13.00	4.93	3.61	1.19
14.00	5.24	3.90	0.71
15.00	5.45	4.10	0.56
16.00	5.62	4.26	0.44
17.00	5.76	4.40	0.38
18.00	5.89	4.52	0.34
19.00	5.99	4.62	0.29
20.00	6.08	4.71	0.25
21.00	6.16	4.78	0.23
22.00	6.24	4.86	0.22
23.00	6.32	4.93	0.22
24.00	**6.39**	**5.00**	0.21
25.00	6.39	5.00	0.00
26.00	6.39	5.00	0.00
27.00	6.39	5.00	0.00
28.00	6.39	5.00	0.00
29.00	6.39	5.00	0.00
30.00	6.39	5.00	0.00
31.00	6.39	5.00	0.00
32.00	6.39	5.00	0.00
33.00	6.39	5.00	0.00
34.00	6.39	5.00	0.00
35.00	6.39	5.00	0.00
36.00	6.39	5.00	0.00
37.00	6.39	5.00	0.00
38.00	6.39	5.00	0.00
39.00	6.39	5.00	0.00
40.00	6.39	5.00	0.00
41.00	6.39	5.00	0.00
42.00	6.39	5.00	0.00
43.00	6.39	5.00	0.00
44.00	6.39	5.00	0.00
45.00	6.39	5.00	0.00
46.00	6.39	5.00	0.00
47.00	6.39	5.00	0.00
48.00	6.39	5.00	0.00
49.00	6.39	5.00	0.00
50.00	6.39	5.00	0.00
51.00	6.39	5.00	0.00
52.00	6.39	5.00	0.00
53.00	6.39	5.00	0.00
54.00	6.39	5.00	0.00
55.00	6.39	5.00	0.00
56.00	6.39	5.00	0.00
57.00	6.39	5.00	0.00
58.00	6.39	5.00	0.00
59.00	6.39	5.00	0.00
60.00	6.39	5.00	0.00
61.00	6.39	5.00	0.00
62.00	6.39	5.00	0.00
63.00	6.39	5.00	0.00
64.00	6.39	5.00	0.00
65.00	6.39	5.00	0.00
66.00	6.39	5.00	0.00
67.00	6.39	5.00	0.00
68.00	6.39	5.00	0.00
69.00	6.39	5.00	0.00
70.00	6.39	5.00	0.00
71.00	6.39	5.00	0.00
72.00	6.39	5.00	0.00

Summary for Subcatchment 29S: Sed. Basin

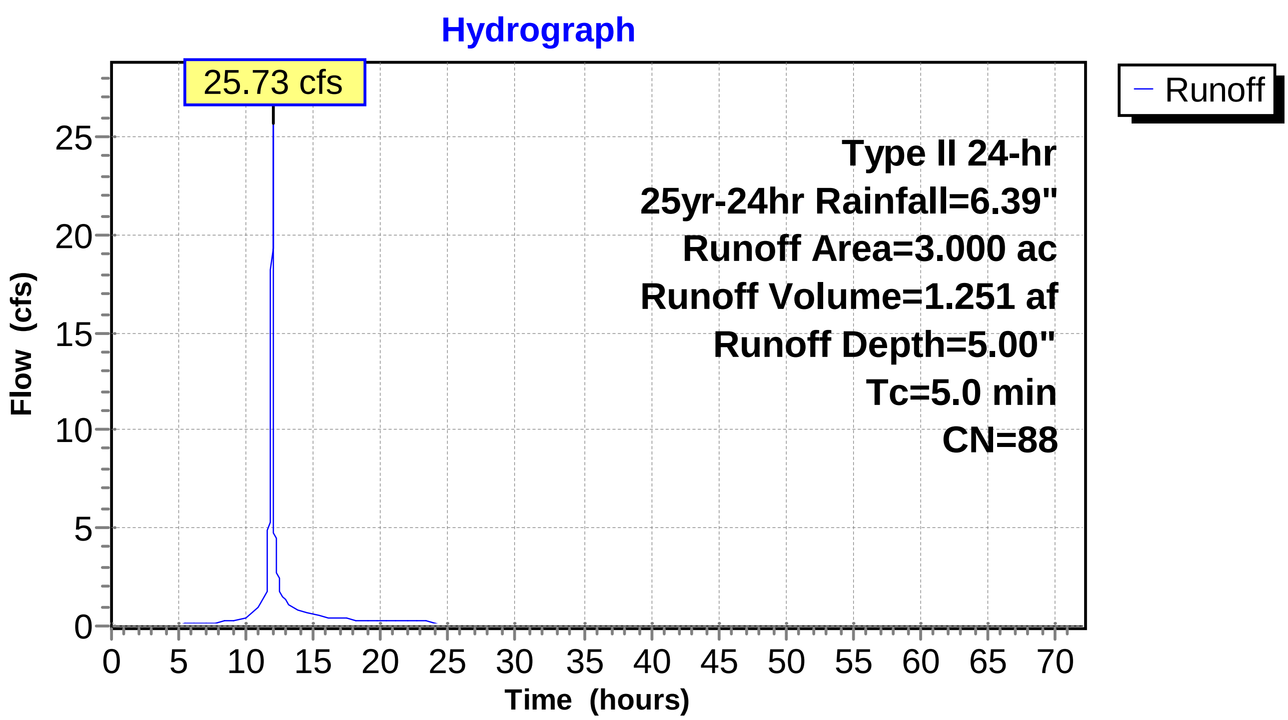
Runoff = 25.73 cfs @ 11.96 hrs, Volume= 1.251 af, Depth= 5.00"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	1.500	98	
*	1.500	78	
	3.000	88	Weighted Average
	1.500		50.00% Pervious Area
	1.500		50.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry, Min. Time of Concentration**

Subcatchment 29S: Sed. Basin



Hydrograph for Subcatchment 29S: Sed. Basin

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.01
5.00	0.40	0.01	0.05
6.00	0.51	0.04	0.09
7.00	0.63	0.08	0.14
8.00	0.77	0.13	0.19
9.00	0.94	0.22	0.33
10.00	1.16	0.35	0.47
11.00	1.50	0.58	**0.95**
12.00	4.24	2.95	**21.48**
13.00	4.93	3.61	1.19
14.00	5.24	3.90	0.71
15.00	5.45	4.10	0.56
16.00	5.62	4.26	0.44
17.00	5.76	4.40	0.38
18.00	5.89	4.52	0.34
19.00	5.99	4.62	0.29
20.00	6.08	4.71	0.25
21.00	6.16	4.78	0.23
22.00	6.24	4.86	0.22
23.00	6.32	4.93	0.22
24.00	**6.39**	**5.00**	0.21
25.00	6.39	5.00	0.00
26.00	6.39	5.00	0.00
27.00	6.39	5.00	0.00
28.00	6.39	5.00	0.00
29.00	6.39	5.00	0.00
30.00	6.39	5.00	0.00
31.00	6.39	5.00	0.00
32.00	6.39	5.00	0.00
33.00	6.39	5.00	0.00
34.00	6.39	5.00	0.00
35.00	6.39	5.00	0.00
36.00	6.39	5.00	0.00
37.00	6.39	5.00	0.00
38.00	6.39	5.00	0.00
39.00	6.39	5.00	0.00
40.00	6.39	5.00	0.00
41.00	6.39	5.00	0.00
42.00	6.39	5.00	0.00
43.00	6.39	5.00	0.00
44.00	6.39	5.00	0.00
45.00	6.39	5.00	0.00
46.00	6.39	5.00	0.00
47.00	6.39	5.00	0.00
48.00	6.39	5.00	0.00
49.00	6.39	5.00	0.00
50.00	6.39	5.00	0.00
51.00	6.39	5.00	0.00
52.00	6.39	5.00	0.00
53.00	6.39	5.00	0.00
54.00	6.39	5.00	0.00
55.00	6.39	5.00	0.00
56.00	6.39	5.00	0.00
57.00	6.39	5.00	0.00
58.00	6.39	5.00	0.00
59.00	6.39	5.00	0.00
60.00	6.39	5.00	0.00
61.00	6.39	5.00	0.00
62.00	6.39	5.00	0.00
63.00	6.39	5.00	0.00
64.00	6.39	5.00	0.00
65.00	6.39	5.00	0.00
66.00	6.39	5.00	0.00
67.00	6.39	5.00	0.00
68.00	6.39	5.00	0.00
69.00	6.39	5.00	0.00
70.00	6.39	5.00	0.00
71.00	6.39	5.00	0.00
72.00	6.39	5.00	0.00

Summary for Subcatchment 30S: North Sed. Basin Culvert

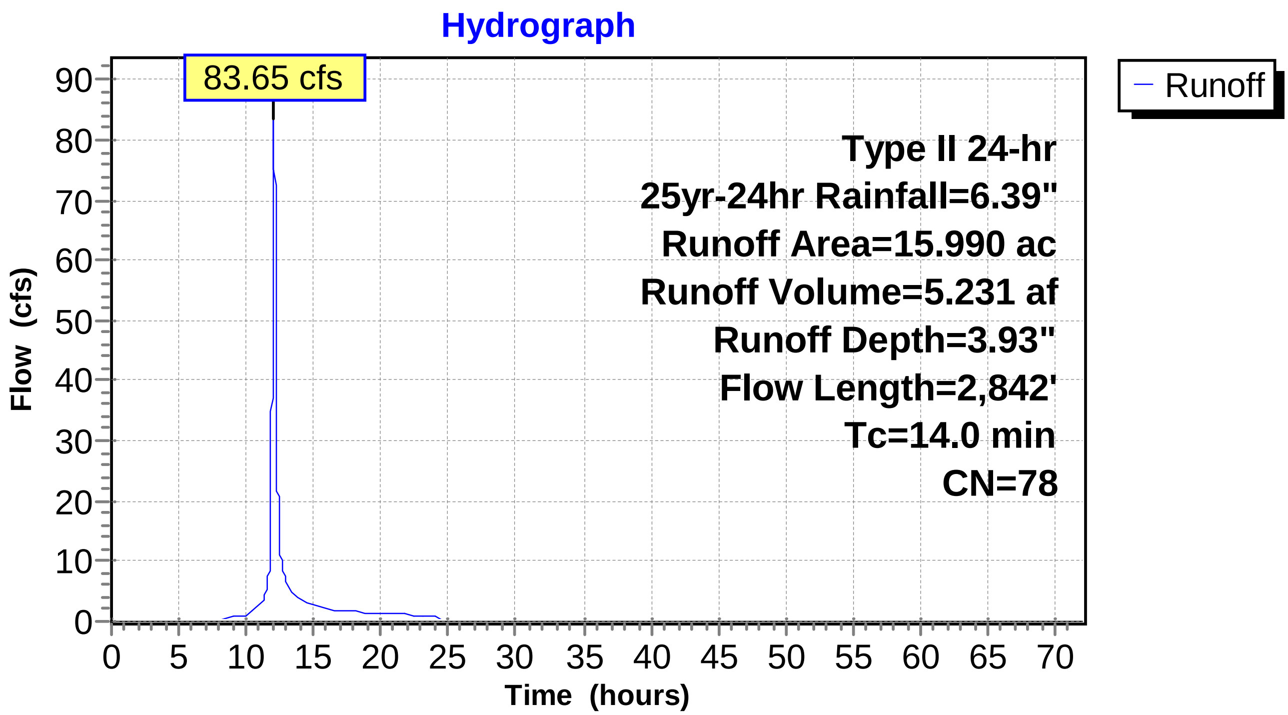
Runoff = 83.65 cfs @ 12.06 hrs, Volume= 5.231 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 30S: North Sed. Basin Culvert



Hydrograph for Subcatchment 30S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.06
8.00	0.77	0.01	0.25
9.00	0.94	0.04	0.62
10.00	1.16	0.10	1.12
11.00	1.50	0.23	2.67
12.00	4.24	2.08	**73.26**
13.00	4.93	2.65	**6.29**
14.00	5.24	2.92	3.64
15.00	5.45	3.10	2.81
16.00	5.62	3.25	2.20
17.00	5.76	3.37	1.90
18.00	5.89	3.48	1.68
19.00	5.99	3.57	1.46
20.00	6.08	3.65	1.23
21.00	6.16	3.72	1.15
22.00	6.24	3.79	1.11
23.00	6.32	3.86	1.07
24.00	**6.39**	**3.93**	1.02
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 31S: South Sed. Basin Culvert

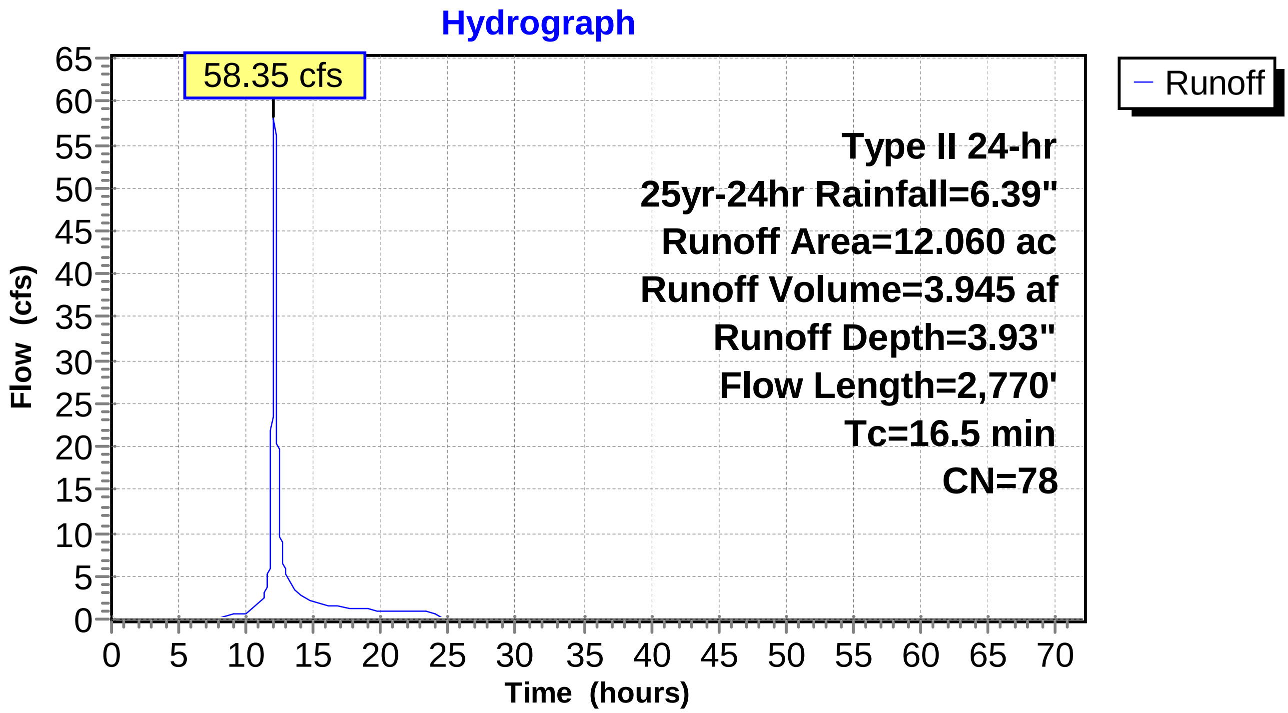
Runoff = 58.35 cfs @ 12.08 hrs, Volume= 3.945 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 31S: South Sed. Basin Culvert



Hydrograph for Subcatchment 31S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.04
8.00	0.77	0.01	0.18
9.00	0.94	0.04	0.45
10.00	1.16	0.10	0.82
11.00	1.50	0.23	1.94
12.00	4.24	2.08	**46.36**
13.00	4.93	2.65	**4.91**
14.00	5.24	2.92	2.80
15.00	5.45	3.10	2.13
16.00	5.62	3.25	1.68
17.00	5.76	3.37	1.44
18.00	5.89	3.48	1.27
19.00	5.99	3.57	1.11
20.00	6.08	3.65	0.94
21.00	6.16	3.72	0.87
22.00	6.24	3.79	0.84
23.00	6.32	3.86	0.81
24.00	**6.39**	**3.93**	0.77
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 32S: North Sed. Basin Culvert

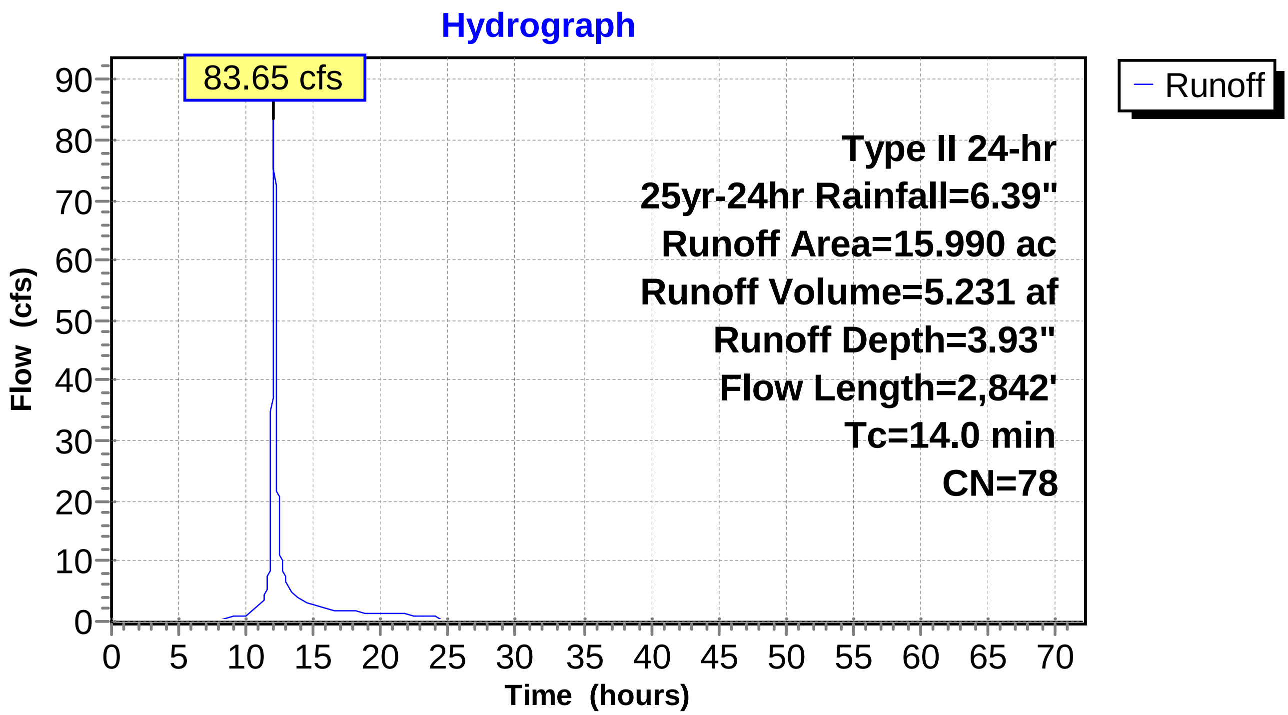
Runoff = 83.65 cfs @ 12.06 hrs, Volume= 5.231 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 32S: North Sed. Basin Culvert



Hydrograph for Subcatchment 32S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.06
8.00	0.77	0.01	0.25
9.00	0.94	0.04	0.62
10.00	1.16	0.10	1.12
11.00	1.50	0.23	2.67
12.00	4.24	2.08	**73.26**
13.00	4.93	2.65	**6.29**
14.00	5.24	2.92	3.64
15.00	5.45	3.10	2.81
16.00	5.62	3.25	2.20
17.00	5.76	3.37	1.90
18.00	5.89	3.48	1.68
19.00	5.99	3.57	1.46
20.00	6.08	3.65	1.23
21.00	6.16	3.72	1.15
22.00	6.24	3.79	1.11
23.00	6.32	3.86	1.07
24.00	**6.39**	**3.93**	1.02
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 33S: South Sed. Basin Culvert

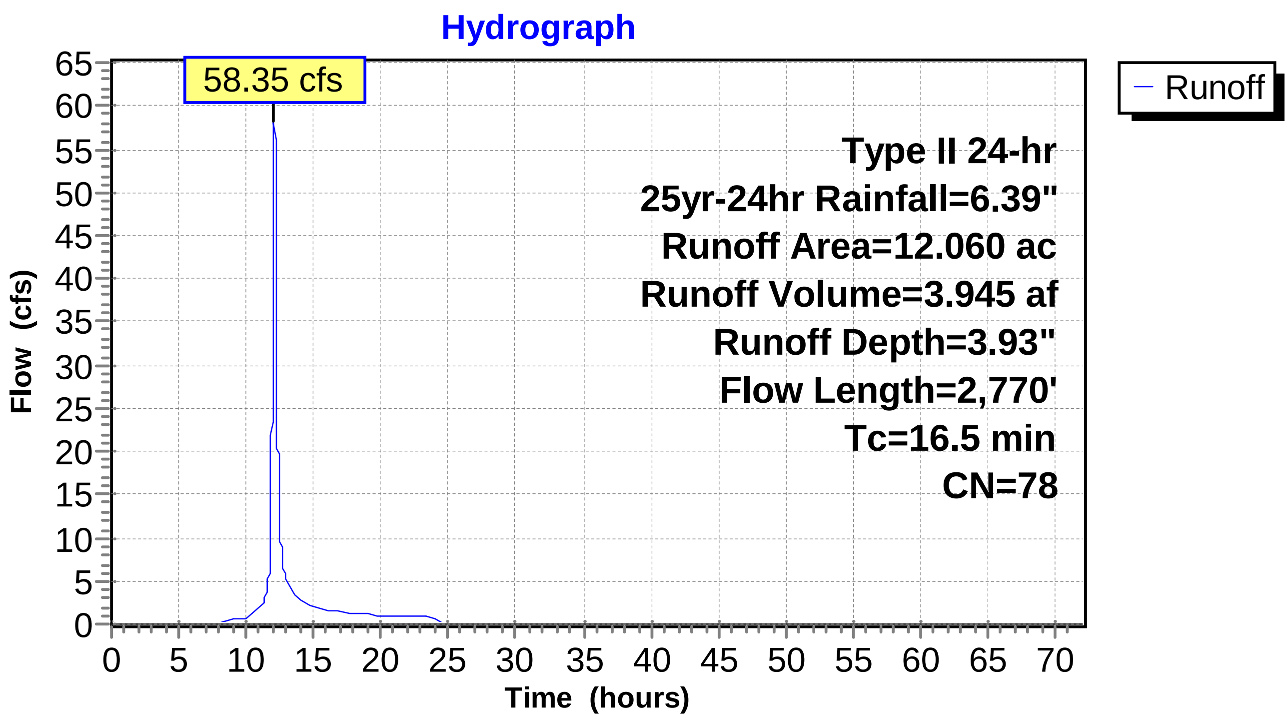
Runoff = 58.35 cfs @ 12.08 hrs, Volume= 3.945 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 33S: South Sed. Basin Culvert



Hydrograph for Subcatchment 33S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)	Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00	54.00	6.39	3.93	0.00
1.00	0.07	0.00	0.00	55.00	6.39	3.93	0.00
2.00	0.14	0.00	0.00	56.00	6.39	3.93	0.00
3.00	0.22	0.00	0.00	57.00	6.39	3.93	0.00
4.00	0.31	0.00	0.00	58.00	6.39	3.93	0.00
5.00	0.40	0.00	0.00	59.00	6.39	3.93	0.00
6.00	0.51	0.00	0.00	60.00	6.39	3.93	0.00
7.00	0.63	0.00	0.04	61.00	6.39	3.93	0.00
8.00	0.77	0.01	0.18	62.00	6.39	3.93	0.00
9.00	0.94	0.04	0.45	63.00	6.39	3.93	0.00
10.00	1.16	0.10	0.82	64.00	6.39	3.93	0.00
11.00	1.50	0.23	1.94	65.00	6.39	3.93	0.00
12.00	4.24	2.08	**46.36**	66.00	6.39	3.93	0.00
13.00	4.93	2.65	**4.91**	67.00	6.39	3.93	0.00
14.00	5.24	2.92	2.80	68.00	6.39	3.93	0.00
15.00	5.45	3.10	2.13	69.00	6.39	3.93	0.00
16.00	5.62	3.25	1.68	70.00	6.39	3.93	0.00
17.00	5.76	3.37	1.44	71.00	6.39	3.93	0.00
18.00	5.89	3.48	1.27	72.00	6.39	3.93	0.00
19.00	5.99	3.57	1.11				
20.00	6.08	3.65	0.94				
21.00	6.16	3.72	0.87				
22.00	6.24	3.79	0.84				
23.00	6.32	3.86	0.81				
24.00	**6.39**	**3.93**	0.77				
25.00	6.39	3.93	0.00				
26.00	6.39	3.93	0.00				
27.00	6.39	3.93	0.00				
28.00	6.39	3.93	0.00				
29.00	6.39	3.93	0.00				
30.00	6.39	3.93	0.00				
31.00	6.39	3.93	0.00				
32.00	6.39	3.93	0.00				
33.00	6.39	3.93	0.00				
34.00	6.39	3.93	0.00				
35.00	6.39	3.93	0.00				
36.00	6.39	3.93	0.00				
37.00	6.39	3.93	0.00				
38.00	6.39	3.93	0.00				
39.00	6.39	3.93	0.00				
40.00	6.39	3.93	0.00				
41.00	6.39	3.93	0.00				
42.00	6.39	3.93	0.00				
43.00	6.39	3.93	0.00				
44.00	6.39	3.93	0.00				
45.00	6.39	3.93	0.00				
46.00	6.39	3.93	0.00				
47.00	6.39	3.93	0.00				
48.00	6.39	3.93	0.00				
49.00	6.39	3.93	0.00				
50.00	6.39	3.93	0.00				
51.00	6.39	3.93	0.00				
52.00	6.39	3.93	0.00				
53.00	6.39	3.93	0.00				

Summary for Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)

Inflow Area = 31.050 ac, 4.83% Impervious, Inflow Depth = 4.03" for 25yr-24hr event
Inflow = 150.51 cfs @ 12.05 hrs, Volume= 10.426 af
Outflow = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af, Atten= 100%, Lag= 0.0 min
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Starting Elev= 439.25' Surf.Area= 0.512 ac Storage= 1.425 af
Peak Elev= 449.67' @ 24.94 hrs Surf.Area= 1.471 ac Storage= 11.851 af (10.426 af above start)
Flood Elev= 450.50' Surf.Area= 1.540 ac Storage= 13.095 af (11.670 af above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

Volume	Invert	Avail.Storage	Storage Description
#1	434.00'	14.687 af	**Sediment Basin (Prismatic)** Listed below (Recalc)

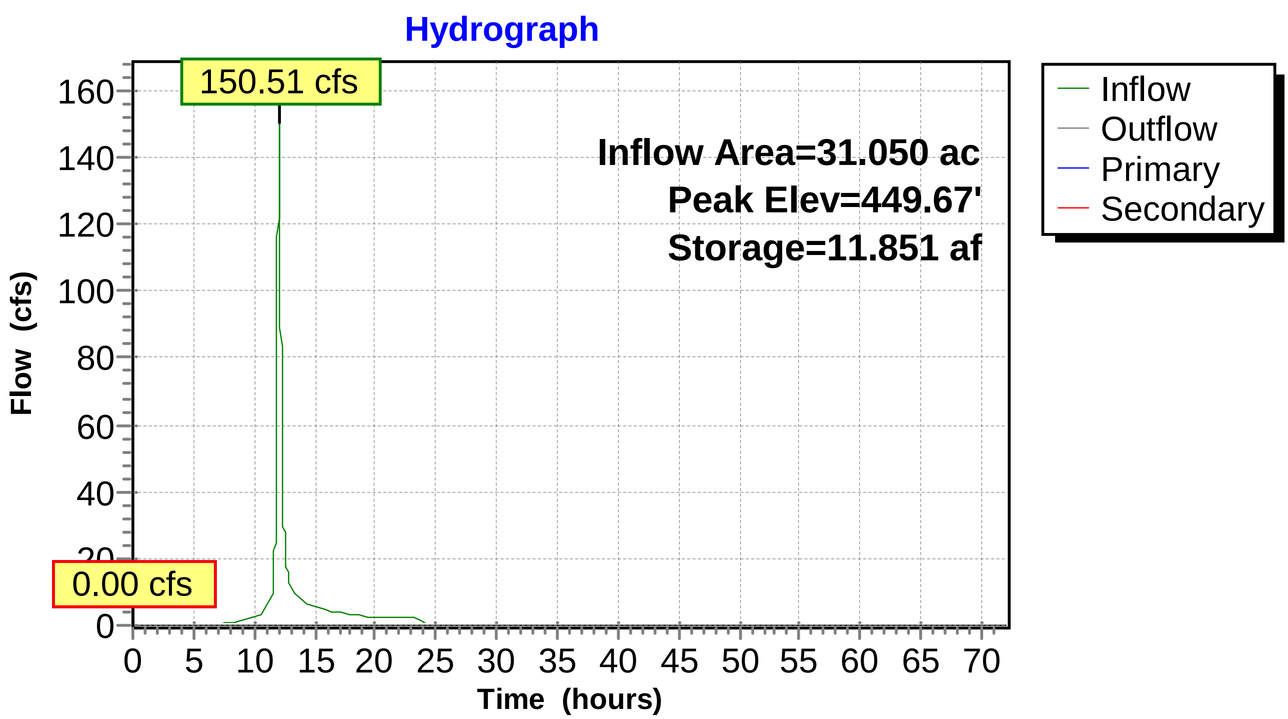
Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
434.00	0.030	0.000	0.000
435.00	0.130	0.080	0.080
436.00	0.220	0.175	0.255
437.00	0.300	0.260	0.515
438.00	0.390	0.345	0.860
439.00	0.490	0.440	1.300
440.00	0.580	0.535	1.835
441.00	0.670	0.625	2.460
442.00	0.770	0.720	3.180
443.00	0.870	0.820	4.000
444.00	0.960	0.915	4.915
445.00	1.060	1.010	5.925
446.00	1.150	1.105	7.030
447.00	1.240	1.195	8.225
448.00	1.330	1.285	9.510
449.00	1.410	1.370	10.880
450.00	1.500	1.455	12.335
451.00	1.580	1.540	13.875
451.50	1.670	0.812	14.687

Device	Routing	Invert	Outlet Devices
#1	Primary	437.75'	**18.0" Round RCP_Round 18"** L= 100.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 437.75' / 435.75' S= 0.0200 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Device 1	450.00'	**48.0" x 48.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#3	Secondary	450.25'	**25.0' long x 22.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

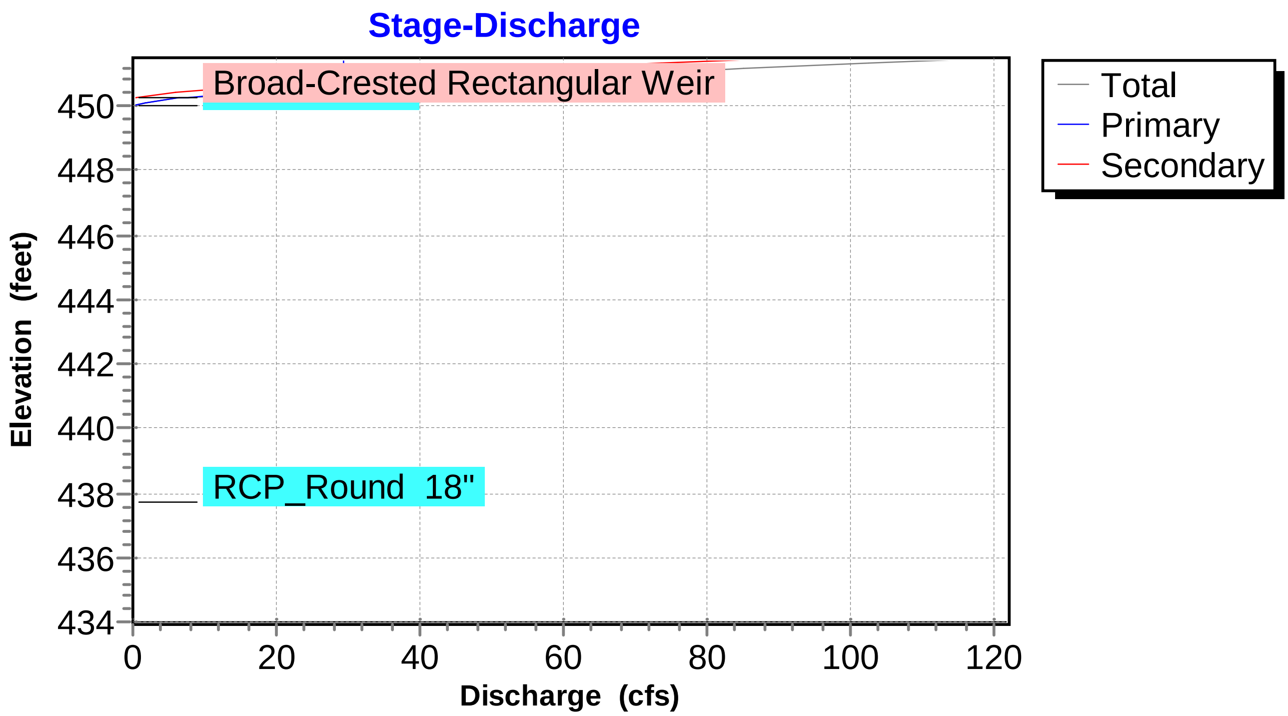
Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=439.25' (Free Discharge)
└─**1=RCP_Round 18"** (Passes 0.00 cfs of 7.37 cfs potential flow)
  └─**2=Orifice/Grate** (Controls 0.00 cfs)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=439.25' (Free Discharge)
└─**3=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)

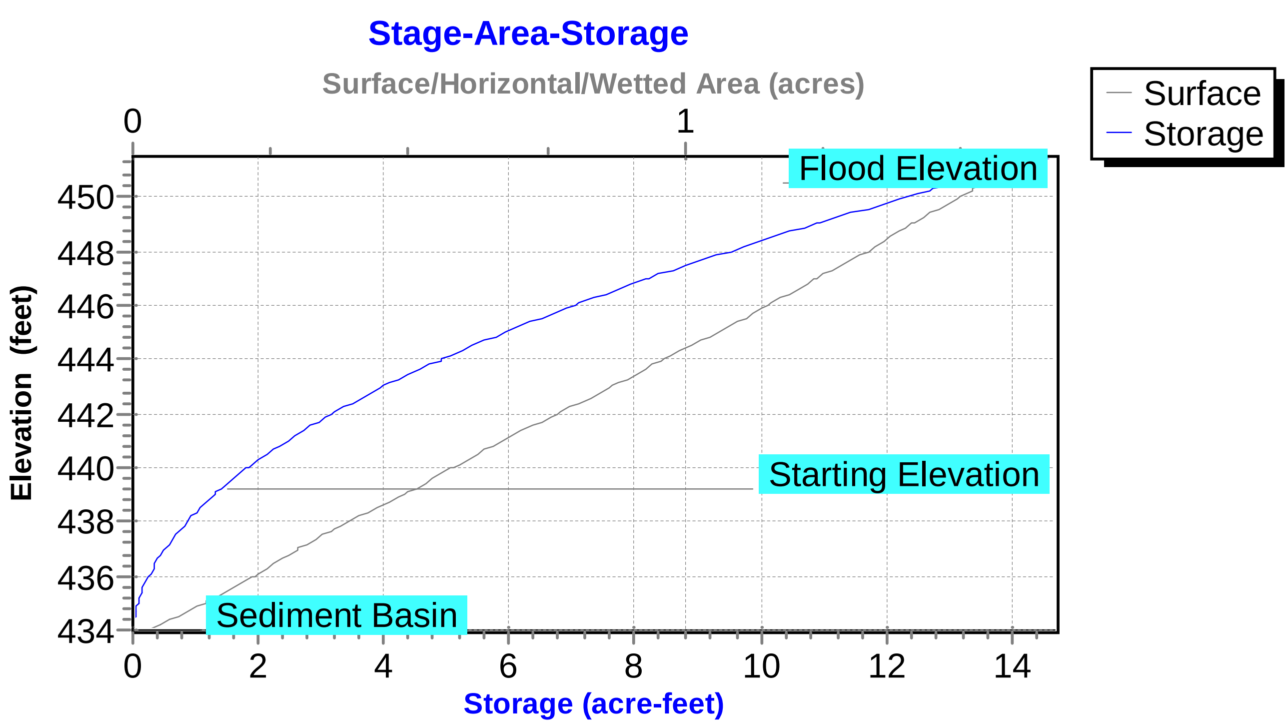
Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)



Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)



Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)



Hydrograph for Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)

Time (hours)	Inflow (cfs)	Storage (acre-feet)	Elevation (feet)	Outflow (cfs)	Primary (cfs)	Secondary (cfs)
0.00	0.00	1.425	439.25	**0.00**	**0.00**	**0.00**
2.00	0.00	1.425	439.25	0.00	0.00	0.00
4.00	0.01	1.425	439.25	0.00	0.00	0.00
6.00	0.09	1.434	439.27	0.00	0.00	0.00
8.00	0.62	1.480	439.35	0.00	0.00	0.00
10.00	2.40	1.710	439.78	0.00	0.00	0.00
12.00	**141.10**	4.300	443.34	0.00	0.00	0.00
14.00	**7.16**	9.093	447.68	0.00	0.00	0.00
16.00	4.31	10.008	448.37	0.00	0.00	0.00
18.00	3.29	10.625	448.82	0.00	0.00	0.00
20.00	2.42	11.097	449.15	0.00	0.00	0.00
22.00	2.17	11.471	449.41	0.00	0.00	0.00
24.00	2.00	**11.816**	**449.65**	0.00	0.00	0.00
26.00	0.00	**11.852**	**449.67**	0.00	0.00	0.00
28.00	0.00	11.852	449.67	0.00	0.00	0.00
30.00	0.00	11.852	449.67	0.00	0.00	0.00
32.00	0.00	11.852	449.67	0.00	0.00	0.00
34.00	0.00	11.852	449.67	0.00	0.00	0.00
36.00	0.00	11.852	449.67	0.00	0.00	0.00
38.00	0.00	11.852	449.67	0.00	0.00	0.00
40.00	0.00	11.852	449.67	0.00	0.00	0.00
42.00	0.00	11.852	449.67	0.00	0.00	0.00
44.00	0.00	11.852	449.67	0.00	0.00	0.00
46.00	0.00	11.852	449.67	0.00	0.00	0.00
48.00	0.00	11.852	449.67	0.00	0.00	0.00
50.00	0.00	11.852	449.67	0.00	0.00	0.00
52.00	0.00	11.852	449.67	0.00	0.00	0.00
54.00	0.00	11.852	449.67	0.00	0.00	0.00
56.00	0.00	11.852	449.67	0.00	0.00	0.00
58.00	0.00	11.852	449.67	0.00	0.00	0.00
60.00	0.00	11.852	449.67	0.00	0.00	0.00
62.00	0.00	11.852	449.67	0.00	0.00	0.00
64.00	0.00	11.852	449.67	0.00	0.00	0.00
66.00	0.00	11.852	449.67	0.00	0.00	0.00
68.00	0.00	11.852	449.67	0.00	0.00	0.00
70.00	0.00	11.852	449.67	0.00	0.00	0.00
72.00	0.00	11.852	449.67	0.00	0.00	0.00

Stage-Discharge for Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)

Elevation (feet)	Discharge (cfs)	Primary (cfs)	Secondary (cfs)
434.00	0.00	0.00	0.00
434.20	0.00	0.00	0.00
434.40	0.00	0.00	0.00
434.60	0.00	0.00	0.00
434.80	0.00	0.00	0.00
435.00	0.00	0.00	0.00
435.20	0.00	0.00	0.00
435.40	0.00	0.00	0.00
435.60	0.00	0.00	0.00
435.80	0.00	0.00	0.00
436.00	0.00	0.00	0.00
436.20	0.00	0.00	0.00
436.40	0.00	0.00	0.00
436.60	0.00	0.00	0.00
436.80	0.00	0.00	0.00
437.00	0.00	0.00	0.00
437.20	0.00	0.00	0.00
437.40	0.00	0.00	0.00
437.60	0.00	0.00	0.00
437.80	0.00	0.00	0.00
438.00	0.00	0.00	0.00
438.20	0.00	0.00	0.00
438.40	0.00	0.00	0.00
438.60	0.00	0.00	0.00
438.80	0.00	0.00	0.00
439.00	0.00	0.00	0.00
439.20	0.00	0.00	0.00
439.40	0.00	0.00	0.00
439.60	0.00	0.00	0.00
439.80	0.00	0.00	0.00
440.00	0.00	0.00	0.00
440.20	0.00	0.00	0.00
440.40	0.00	0.00	0.00
440.60	0.00	0.00	0.00
440.80	0.00	0.00	0.00
441.00	0.00	0.00	0.00
441.20	0.00	0.00	0.00
441.40	0.00	0.00	0.00
441.60	0.00	0.00	0.00
441.80	0.00	0.00	0.00
442.00	0.00	0.00	0.00
442.20	0.00	0.00	0.00
442.40	0.00	0.00	0.00
442.60	0.00	0.00	0.00
442.80	0.00	0.00	0.00
443.00	0.00	0.00	0.00
443.20	0.00	0.00	0.00
443.40	0.00	0.00	0.00
443.60	0.00	0.00	0.00
443.80	0.00	0.00	0.00
444.00	0.00	0.00	0.00
444.20	0.00	0.00	0.00
444.40	0.00	0.00	0.00
444.60	0.00	0.00	0.00
444.80	0.00	0.00	0.00
445.00	0.00	0.00	0.00
445.20	0.00	0.00	0.00
445.40	0.00	0.00	0.00
445.60	0.00	0.00	0.00
445.80	0.00	0.00	0.00
446.00	0.00	0.00	0.00
446.20	0.00	0.00	0.00
446.40	0.00	0.00	0.00
446.60	0.00	0.00	0.00
446.80	0.00	0.00	0.00
447.00	0.00	0.00	0.00
447.20	0.00	0.00	0.00
447.40	0.00	0.00	0.00
447.60	0.00	0.00	0.00
447.80	0.00	0.00	0.00
448.00	0.00	0.00	0.00
448.20	0.00	0.00	0.00
448.40	0.00	0.00	0.00
448.60	0.00	0.00	0.00
448.80	0.00	0.00	0.00
449.00	0.00	0.00	0.00
449.20	0.00	0.00	0.00
449.40	0.00	0.00	0.00
449.60	0.00	0.00	0.00
449.80	0.00	0.00	0.00
450.00	0.00	0.00	0.00
450.20	4.68	4.68	0.00
450.40	17.13	13.24	3.89
450.60	38.27	24.32	13.95
450.80	56.14	28.61	27.53
451.00	71.93	28.82	43.11
451.20	89.97	29.03	60.94
451.40	**110.55**	**29.24**	**81.32**

Stage-Area-Storage for Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)

Elevation (feet)	Surface (acres)	Storage (acre-feet)	Elevation (feet)	Surface (acres)	Storage (acre-feet)
434.00	0.030	0.000	444.80	1.040	5.715
434.20	0.050	0.008	445.00	1.060	5.925
434.40	0.070	0.020	445.20	1.078	6.139
434.60	0.090	0.036	445.40	1.096	6.356
434.80	0.110	0.056	445.60	1.114	6.577
435.00	0.130	0.080	445.80	1.132	6.802
435.20	0.148	0.108	446.00	1.150	7.030
435.40	0.166	0.139	446.20	1.168	7.262
435.60	0.184	0.174	446.40	1.186	7.497
435.80	0.202	0.213	446.60	1.204	7.736
436.00	0.220	0.255	446.80	1.222	7.979
436.20	0.236	0.301	447.00	1.240	8.225
436.40	0.252	0.349	447.20	1.258	8.475
436.60	0.268	0.401	447.40	1.276	8.728
436.80	0.284	0.457	447.60	1.294	8.985
437.00	0.300	0.515	447.80	1.312	9.246
437.20	0.318	0.577	448.00	1.330	9.510
437.40	0.336	0.642	448.20	1.346	9.778
437.60	0.354	0.711	448.40	1.362	10.048
437.80	0.372	0.784	448.60	1.378	10.322
438.00	0.390	0.860	448.80	1.394	10.600
438.20	0.410	0.940	449.00	1.410	10.880
438.40	0.430	1.024	449.20	1.428	11.164
438.60	0.450	1.112	449.40	1.446	11.451
438.80	0.470	1.204	449.60	1.464	11.742
439.00	0.490	1.300	449.80	1.482	12.037
439.20	0.508	1.400	450.00	1.500	12.335
439.40	0.526	1.503	450.20	1.516	12.637
439.60	0.544	1.610	450.40	1.532	12.941
439.80	0.562	1.721	450.60	1.548	13.249
440.00	0.580	1.835	450.80	1.564	13.561
440.20	0.598	1.953	451.00	1.580	13.875
440.40	0.616	2.074	451.20	1.616	14.195
440.60	0.634	2.199	451.40	**1.652**	**14.521**
440.80	0.652	2.328			
441.00	0.670	2.460			
441.20	0.690	2.596			
441.40	0.710	2.736			
441.60	0.730	2.880			
441.80	0.750	3.028			
442.00	0.770	3.180			
442.20	0.790	3.336			
442.40	0.810	3.496			
442.60	0.830	3.660			
442.80	0.850	3.828			
443.00	0.870	4.000			
443.20	0.888	4.176			
443.40	0.906	4.355			
443.60	0.924	4.538			
443.80	0.942	4.725			
444.00	0.960	4.915			
444.20	0.980	5.109			
444.40	1.000	5.307			
444.60	1.020	5.509			

Summary for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Inflow Area = 31.050 ac, 4.83% Impervious, Inflow Depth = 4.03" for 25yr-24hr event
Inflow = 150.51 cfs @ 12.05 hrs, Volume= 10.426 af
Outflow = 3.96 cfs @ 16.43 hrs, Volume= 9.885 af, Atten= 97%, Lag= 263.0 min
Primary = 3.96 cfs @ 16.43 hrs, Volume= 9.885 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Starting Elev= 439.25' Surf.Area= 0.512 ac Storage= 1.425 af
Peak Elev= 447.32' @ 16.43 hrs Surf.Area= 1.269 ac Storage= 8.622 af (7.197 af above start)
Flood Elev= 450.50' Surf.Area= 1.540 ac Storage= 13.095 af (11.670 af above start)

Plug-Flow detention time= 1,350.0 min calculated for 8.459 af (81% of inflow)
Center-of-Mass det. time= 1,087.9 min (1,905.4 - 817.5)

Volume	Invert	Avail.Storage	Storage Description
#1	434.00'	14.687 af	**Sediment Basin (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
434.00	0.030	0.000	0.000
435.00	0.130	0.080	0.080
436.00	0.220	0.175	0.255
437.00	0.300	0.260	0.515
438.00	0.390	0.345	0.860
439.00	0.490	0.440	1.300
440.00	0.580	0.535	1.835
441.00	0.670	0.625	2.460
442.00	0.770	0.720	3.180
443.00	0.870	0.820	4.000
444.00	0.960	0.915	4.915
445.00	1.060	1.010	5.925
446.00	1.150	1.105	7.030
447.00	1.240	1.195	8.225
448.00	1.330	1.285	9.510
449.00	1.410	1.370	10.880
450.00	1.500	1.455	12.335
451.00	1.580	1.540	13.875
451.50	1.670	0.812	14.687

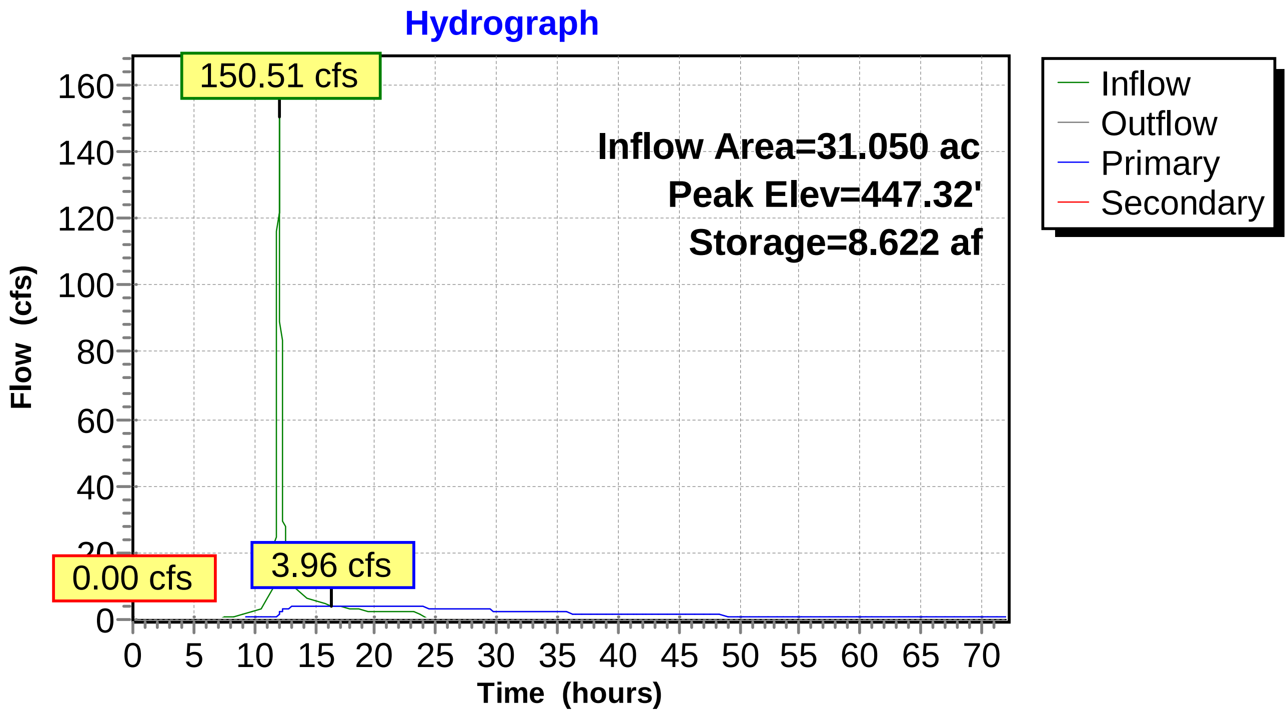
Device	Routing	Invert	Outlet Devices
#1	Primary	438.75'	**18.0" Round RCP_Round 18"** L= 100.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 438.75' / 436.75' S= 0.0200 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Device 1	439.25'	**0.734 cfs Constant Flow/Skimmer** Phase-In= 0.50'
#3	Device 1	441.25'	**1.5" Vert. Orifice/Grate X 5.00 columns** X 8 rows with 12.0" cc spacing C= 0.600
#4	Device 1	450.00'	**48.0" x 48.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

#5	Secondary	450.25'	**25.0' long x 22.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

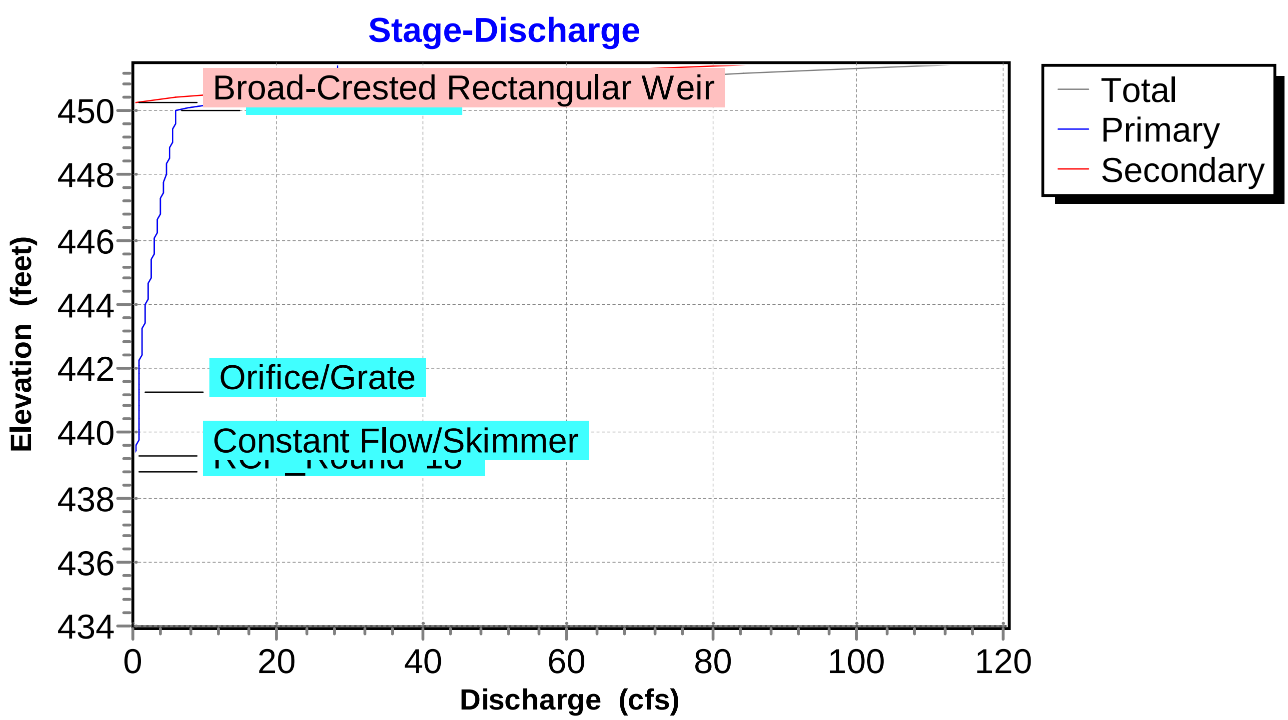
Primary OutFlow Max=3.97 cfs @ 16.43 hrs HW=447.32' (Free Discharge)
└─**1=RCP_Round 18"** (Passes 3.97 cfs of 23.40 cfs potential flow)
  ├─**2=Constant Flow/Skimmer** (Constant Controls 0.73 cfs)
  ├─**3=Orifice/Grate** (Orifice Controls 3.23 cfs @ 8.05 fps)
  └─**4=Orifice/Grate** (Controls 0.00 cfs)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=439.25' (Free Discharge)
└─**5=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)

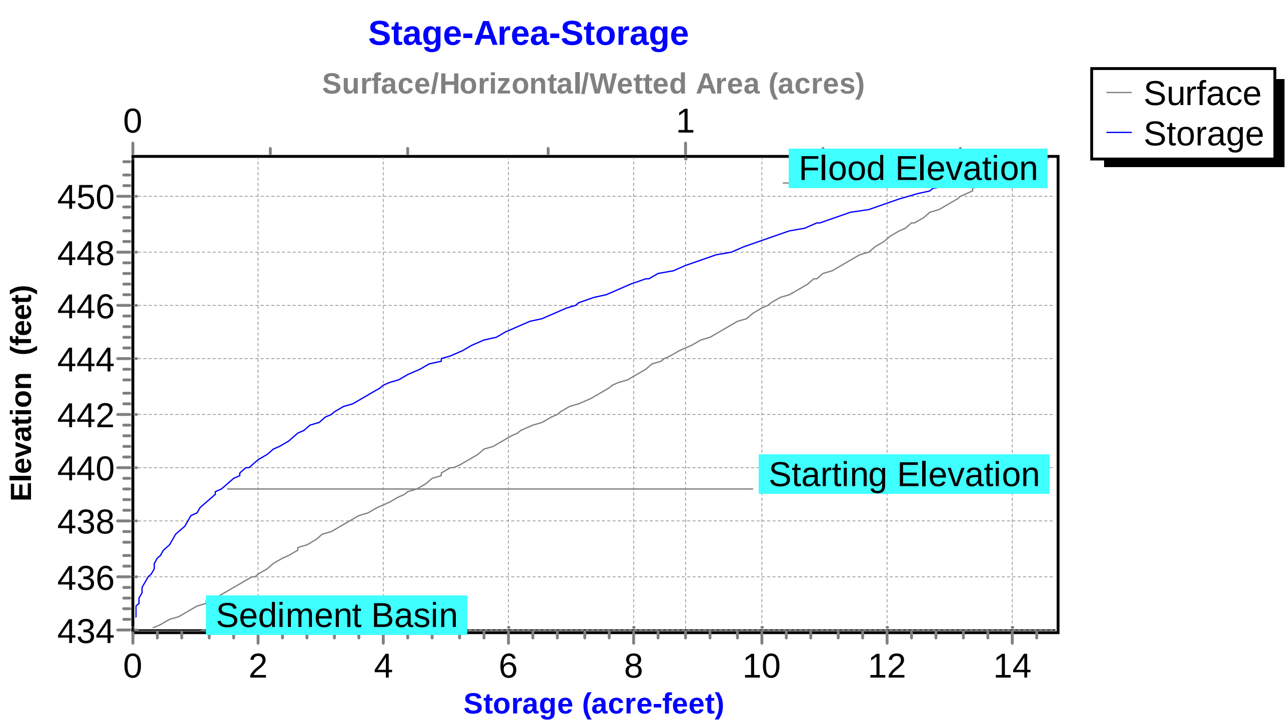
Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Hydrograph for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Time (hours)	Inflow (cfs)	Storage (acre-feet)	Elevation (feet)	Outflow (cfs)	Primary (cfs)	Secondary (cfs)
0.00	0.00	1.425	439.25	0.00	0.00	**0.00**
2.00	0.00	1.425	439.25	0.00	0.00	0.00
4.00	0.01	1.425	439.25	0.00	0.00	0.00
6.00	0.09	1.432	439.26	0.02	0.02	0.00
8.00	0.62	1.470	439.34	0.12	0.12	0.00
10.00	2.40	1.646	439.67	0.61	0.61	0.00
12.00	**141.10**	4.112	443.13	1.40	1.40	0.00
14.00	**7.16**	8.347	447.10	3.82	3.82	0.00
16.00	4.31	**8.617**	**447.31**	**3.96**	**3.96**	0.00
18.00	3.29	**8.580**	**447.28**	**3.93**	**3.93**	0.00
20.00	2.42	8.409	447.15	3.85	3.85	0.00
22.00	2.17	8.157	446.94	3.73	3.73	0.00
24.00	2.00	7.897	446.73	3.59	3.59	0.00
26.00	0.00	7.366	446.29	3.22	3.22	0.00
28.00	0.00	6.855	445.85	2.97	2.97	0.00
30.00	0.00	6.386	445.43	2.69	2.69	0.00
32.00	0.00	5.966	445.04	2.43	2.43	0.00
34.00	0.00	5.579	444.67	2.25	2.25	0.00
36.00	0.00	5.227	444.32	2.00	2.00	0.00
38.00	0.00	4.910	444.00	1.85	1.85	0.00
40.00	0.00	4.616	443.68	1.71	1.71	0.00
42.00	0.00	4.346	443.39	1.54	1.54	0.00
44.00	0.00	4.106	443.12	1.40	1.40	0.00
46.00	0.00	3.882	442.86	1.32	1.32	0.00
48.00	0.00	3.670	442.61	1.23	1.23	0.00
50.00	0.00	3.476	442.38	1.11	1.11	0.00
52.00	0.00	3.303	442.16	1.01	1.01	0.00
54.00	0.00	3.140	441.95	0.97	0.97	0.00
56.00	0.00	2.983	441.74	0.93	0.93	0.00
58.00	0.00	2.834	441.54	0.87	0.87	0.00
60.00	0.00	2.697	441.34	0.79	0.79	0.00
62.00	0.00	2.573	441.17	0.73	0.73	0.00
64.00	0.00	2.452	440.99	0.73	0.73	0.00
66.00	0.00	2.331	440.80	0.73	0.73	0.00
68.00	0.00	2.209	440.62	0.73	0.73	0.00
70.00	0.00	2.088	440.42	0.73	0.73	0.00
72.00	0.00	1.967	440.22	0.73	0.73	0.00

Stage-Discharge for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Elevation (feet)	Discharge (cfs)	Primary (cfs)	Secondary (cfs)	Elevation (feet)	Discharge (cfs)	Primary (cfs)	Secondary (cfs)
434.00	0.00	0.00	0.00	444.80	2.32	2.32	0.00
434.20	0.00	0.00	0.00	445.00	2.41	2.41	0.00
434.40	0.00	0.00	0.00	445.20	2.50	2.50	0.00
434.60	0.00	0.00	0.00	445.40	2.67	2.67	0.00
434.80	0.00	0.00	0.00	445.60	2.82	2.82	0.00
435.00	0.00	0.00	0.00	445.80	2.94	2.94	0.00
435.20	0.00	0.00	0.00	446.00	3.05	3.05	0.00
435.40	0.00	0.00	0.00	446.20	3.16	3.16	0.00
435.60	0.00	0.00	0.00	446.40	3.34	3.34	0.00
435.80	0.00	0.00	0.00	446.60	3.50	3.50	0.00
436.00	0.00	0.00	0.00	446.80	3.64	3.64	0.00
436.20	0.00	0.00	0.00	447.00	3.76	3.76	0.00
436.40	0.00	0.00	0.00	447.20	3.87	3.87	0.00
436.60	0.00	0.00	0.00	447.40	4.07	4.07	0.00
436.80	0.00	0.00	0.00	447.60	4.24	4.24	0.00
437.00	0.00	0.00	0.00	447.80	4.39	4.39	0.00
437.20	0.00	0.00	0.00	448.00	4.52	4.52	0.00
437.40	0.00	0.00	0.00	448.20	4.65	4.65	0.00
437.60	0.00	0.00	0.00	448.40	4.85	4.85	0.00
437.80	0.00	0.00	0.00	448.60	5.04	5.04	0.00
438.00	0.00	0.00	0.00	448.80	5.20	5.20	0.00
438.20	0.00	0.00	0.00	449.00	5.34	5.34	0.00
438.40	0.00	0.00	0.00	449.20	5.48	5.48	0.00
438.60	0.00	0.00	0.00	449.40	5.61	5.61	0.00
438.80	0.00	0.00	0.00	449.60	5.73	5.73	0.00
439.00	0.00	0.00	0.00	449.80	5.85	5.85	0.00
439.20	0.00	0.00	0.00	450.00	5.97	5.97	0.00
439.40	0.22	0.22	0.00	450.20	10.76	10.76	0.00
439.60	0.51	0.51	0.00	450.40	23.32	19.43	3.89
439.80	0.73	0.73	0.00	450.60	41.27	27.32	13.95
440.00	0.73	0.73	0.00	450.80	55.07	27.54	27.53
440.20	0.73	0.73	0.00	451.00	70.87	27.75	43.11
440.40	0.73	0.73	0.00	451.20	88.91	27.97	60.94
440.60	0.73	0.73	0.00	451.40	**109.50**	**28.19**	**81.32**
440.80	0.73	0.73	0.00				
441.00	0.73	0.73	0.00				
441.20	0.73	0.73	0.00				
441.40	0.82	0.82	0.00				
441.60	0.89	0.89	0.00				
441.80	0.94	0.94	0.00				
442.00	0.98	0.98	0.00				
442.20	1.01	1.01	0.00				
442.40	1.13	1.13	0.00				
442.60	1.23	1.23	0.00				
442.80	1.30	1.30	0.00				
443.00	1.36	1.36	0.00				
443.20	1.42	1.42	0.00				
443.40	1.56	1.56	0.00				
443.60	1.67	1.67	0.00				
443.80	1.77	1.77	0.00				
444.00	1.85	1.85	0.00				
444.20	1.92	1.92	0.00				
444.40	2.08	2.08	0.00				
444.60	2.21	2.21	0.00				

Stage-Area-Storage for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Elevation (feet)	Surface (acres)	Storage (acre-feet)	Elevation (feet)	Surface (acres)	Storage (acre-feet)
434.00	0.030	0.000	444.80	1.040	5.715
434.20	0.050	0.008	445.00	1.060	5.925
434.40	0.070	0.020	445.20	1.078	6.139
434.60	0.090	0.036	445.40	1.096	6.356
434.80	0.110	0.056	445.60	1.114	6.577
435.00	0.130	0.080	445.80	1.132	6.802
435.20	0.148	0.108	446.00	1.150	7.030
435.40	0.166	0.139	446.20	1.168	7.262
435.60	0.184	0.174	446.40	1.186	7.497
435.80	0.202	0.213	446.60	1.204	7.736
436.00	0.220	0.255	446.80	1.222	7.979
436.20	0.236	0.301	447.00	1.240	8.225
436.40	0.252	0.349	447.20	1.258	8.475
436.60	0.268	0.401	447.40	1.276	8.728
436.80	0.284	0.457	447.60	1.294	8.985
437.00	0.300	0.515	447.80	1.312	9.246
437.20	0.318	0.577	448.00	1.330	9.510
437.40	0.336	0.642	448.20	1.346	9.778
437.60	0.354	0.711	448.40	1.362	10.048
437.80	0.372	0.784	448.60	1.378	10.322
438.00	0.390	0.860	448.80	1.394	10.600
438.20	0.410	0.940	449.00	1.410	10.880
438.40	0.430	1.024	449.20	1.428	11.164
438.60	0.450	1.112	449.40	1.446	11.451
438.80	0.470	1.204	449.60	1.464	11.742
439.00	0.490	1.300	449.80	1.482	12.037
439.20	0.508	1.400	450.00	1.500	12.335
439.40	0.526	1.503	450.20	1.516	12.637
439.60	0.544	1.610	450.40	1.532	12.941
439.80	0.562	1.721	450.60	1.548	13.249
440.00	0.580	1.835	450.80	1.564	13.561
440.20	0.598	1.953	451.00	1.580	13.875
440.40	0.616	2.074	451.20	1.616	14.195
440.60	0.634	2.199	451.40	**1.652**	**14.521**
440.80	0.652	2.328			
441.00	0.670	2.460			
441.20	0.690	2.596			
441.40	0.710	2.736			
441.60	0.730	2.880			
441.80	0.750	3.028			
442.00	0.770	3.180			
442.20	0.790	3.336			
442.40	0.810	3.496			
442.60	0.830	3.660			
442.80	0.850	3.828			
443.00	0.870	4.000			
443.20	0.888	4.176			
443.40	0.906	4.355			
443.60	0.924	4.538			
443.80	0.942	4.725			
444.00	0.960	4.915			
444.20	0.980	5.109			
444.40	1.000	5.307			
444.60	1.020	5.509			

Summary for Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)

Inflow Area = 31.050 ac, 4.83% Impervious, Inflow Depth = 4.03" for 25yr-24hr event
Inflow = 150.51 cfs @ 12.05 hrs, Volume= 10.426 af
Outflow = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af, Atten= 100%, Lag= 0.0 min
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Starting Elev= 439.25' Surf.Area= 0.512 ac Storage= 1.425 af
Peak Elev= 449.67' @ 24.94 hrs Surf.Area= 1.471 ac Storage= 11.851 af (10.426 af above start)
Flood Elev= 450.50' Surf.Area= 1.540 ac Storage= 13.095 af (11.670 af above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

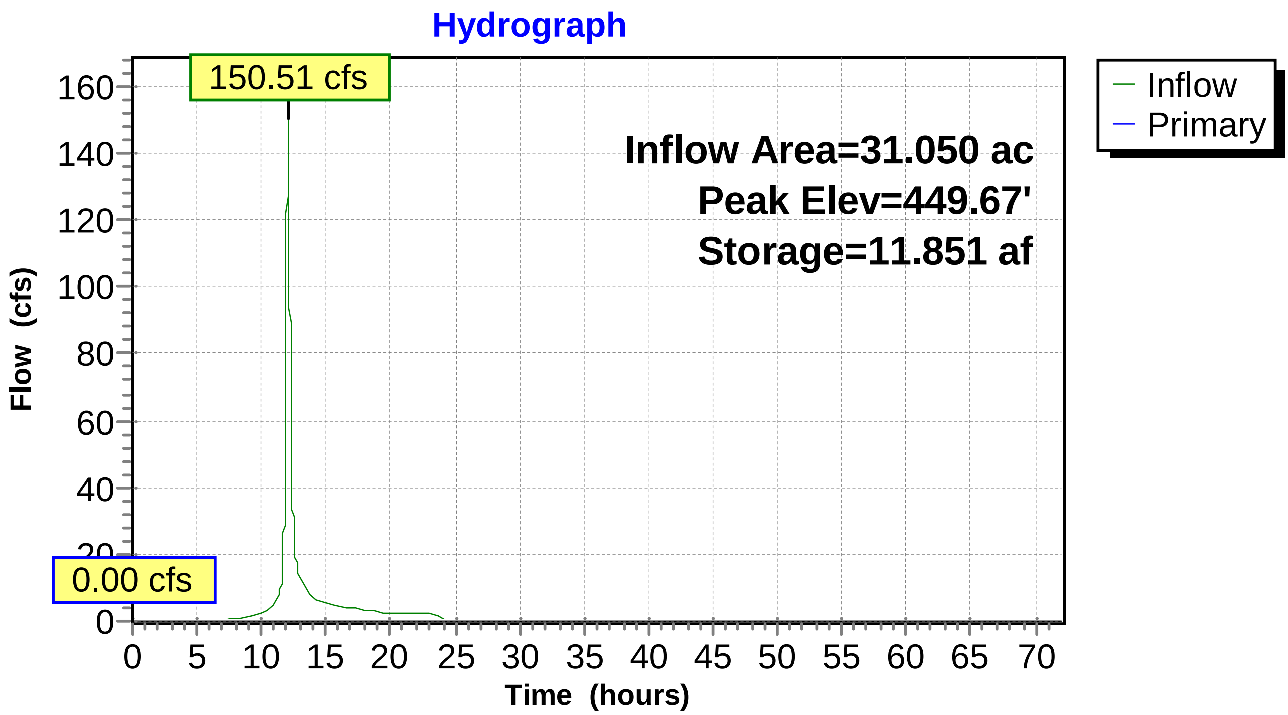
Volume	Invert	Avail.Storage	Storage Description
#1	434.00'	14.687 af	**Sediment Basin (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
434.00	0.030	0.000	0.000
435.00	0.130	0.080	0.080
436.00	0.220	0.175	0.255
437.00	0.300	0.260	0.515
438.00	0.390	0.345	0.860
439.00	0.490	0.440	1.300
440.00	0.580	0.535	1.835
441.00	0.670	0.625	2.460
442.00	0.770	0.720	3.180
443.00	0.870	0.820	4.000
444.00	0.960	0.915	4.915
445.00	1.060	1.010	5.925
446.00	1.150	1.105	7.030
447.00	1.240	1.195	8.225
448.00	1.330	1.285	9.510
449.00	1.410	1.370	10.880
450.00	1.500	1.455	12.335
451.00	1.580	1.540	13.875
451.50	1.670	0.812	14.687

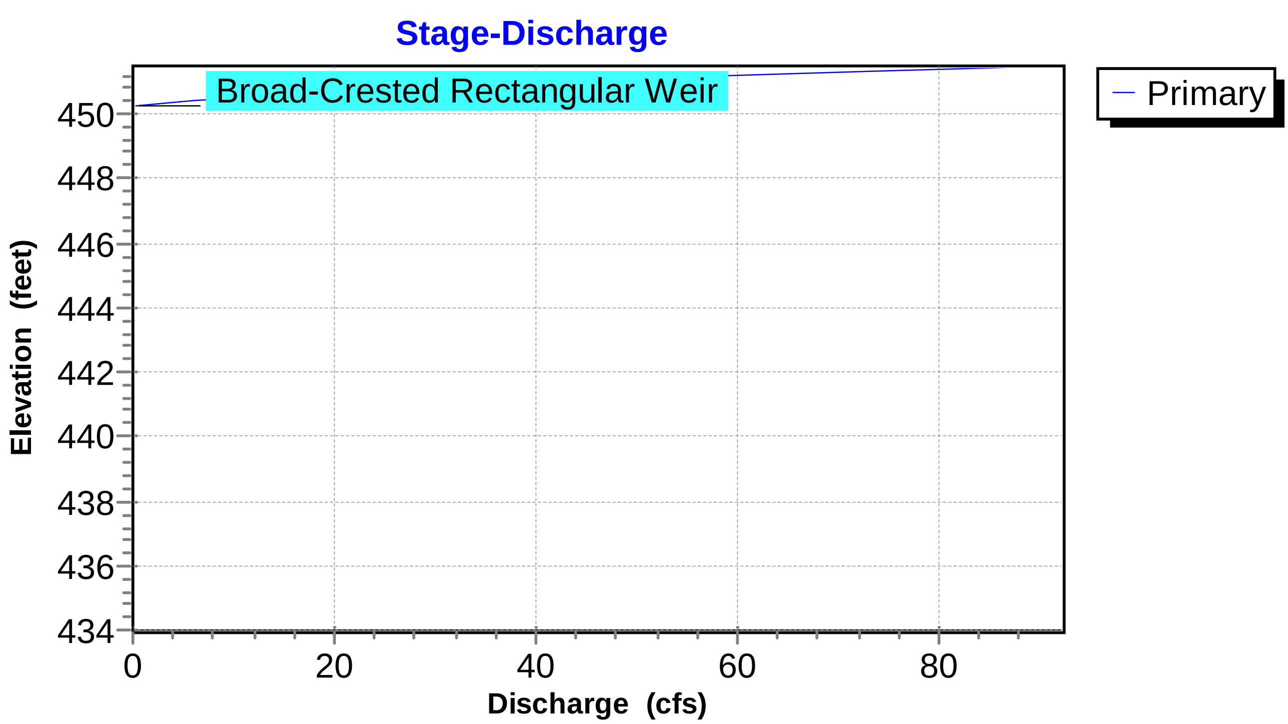
Device	Routing	Invert	Outlet Devices
#1	Primary	450.25'	**25.0' long x 22.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=439.25' (Free Discharge)
└**1=Broad-Crested Rectangular Weir** (Controls 0.00 cfs)

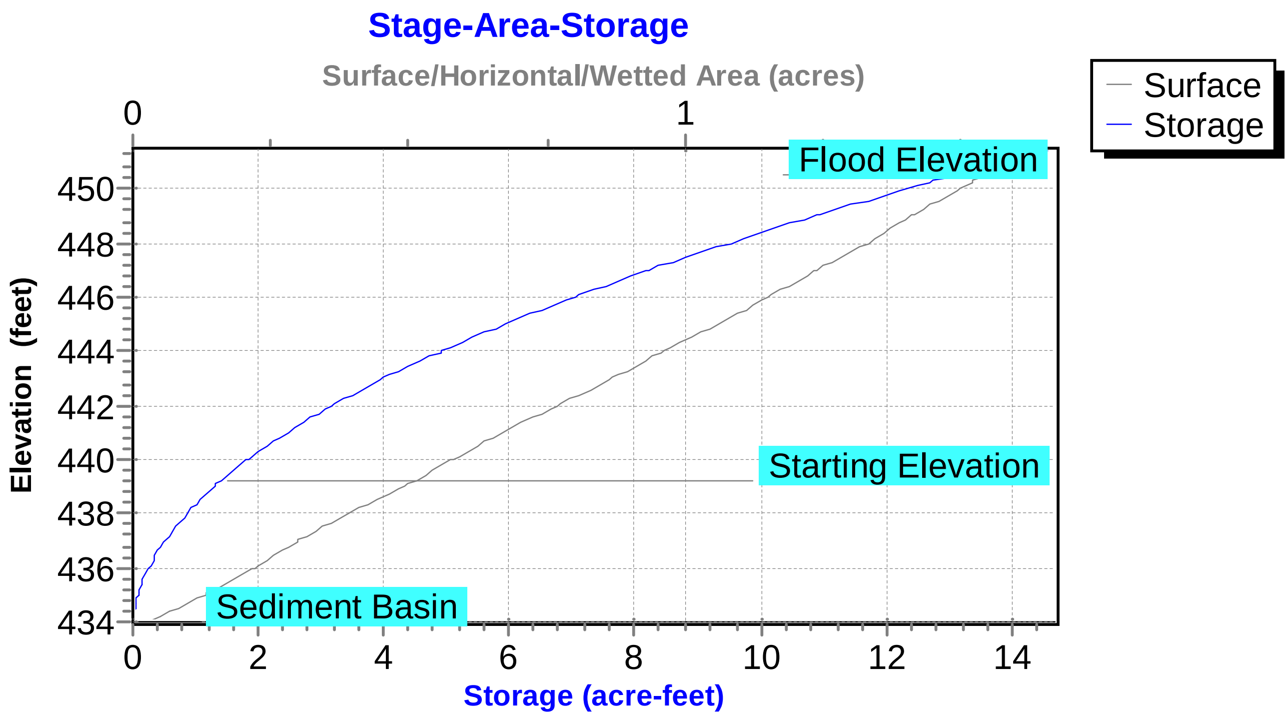
Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)



Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)



Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)



Hydrograph for Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)

Time (hours)	Inflow (cfs)	Storage (acre-feet)	Elevation (feet)	Primary (cfs)
0.00	0.00	1.425	439.25	**0.00**
2.00	0.00	1.425	439.25	0.00
4.00	0.01	1.425	439.25	0.00
6.00	0.09	1.434	439.27	0.00
8.00	0.62	1.480	439.35	0.00
10.00	2.40	1.710	439.78	0.00
12.00	**141.10**	4.300	443.34	0.00
14.00	**7.16**	9.093	447.68	0.00
16.00	4.31	10.008	448.37	0.00
18.00	3.29	10.625	448.82	0.00
20.00	2.42	11.097	449.15	0.00
22.00	2.17	11.471	449.41	0.00
24.00	2.00	**11.816**	**449.65**	0.00
26.00	0.00	**11.852**	**449.67**	0.00
28.00	0.00	11.852	449.67	0.00
30.00	0.00	11.852	449.67	0.00
32.00	0.00	11.852	449.67	0.00
34.00	0.00	11.852	449.67	0.00
36.00	0.00	11.852	449.67	0.00
38.00	0.00	11.852	449.67	0.00
40.00	0.00	11.852	449.67	0.00
42.00	0.00	11.852	449.67	0.00
44.00	0.00	11.852	449.67	0.00
46.00	0.00	11.852	449.67	0.00
48.00	0.00	11.852	449.67	0.00
50.00	0.00	11.852	449.67	0.00
52.00	0.00	11.852	449.67	0.00
54.00	0.00	11.852	449.67	0.00
56.00	0.00	11.852	449.67	0.00
58.00	0.00	11.852	449.67	0.00
60.00	0.00	11.852	449.67	0.00
62.00	0.00	11.852	449.67	0.00
64.00	0.00	11.852	449.67	0.00
66.00	0.00	11.852	449.67	0.00
68.00	0.00	11.852	449.67	0.00
70.00	0.00	11.852	449.67	0.00
72.00	0.00	11.852	449.67	0.00

Stage-Discharge for Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)

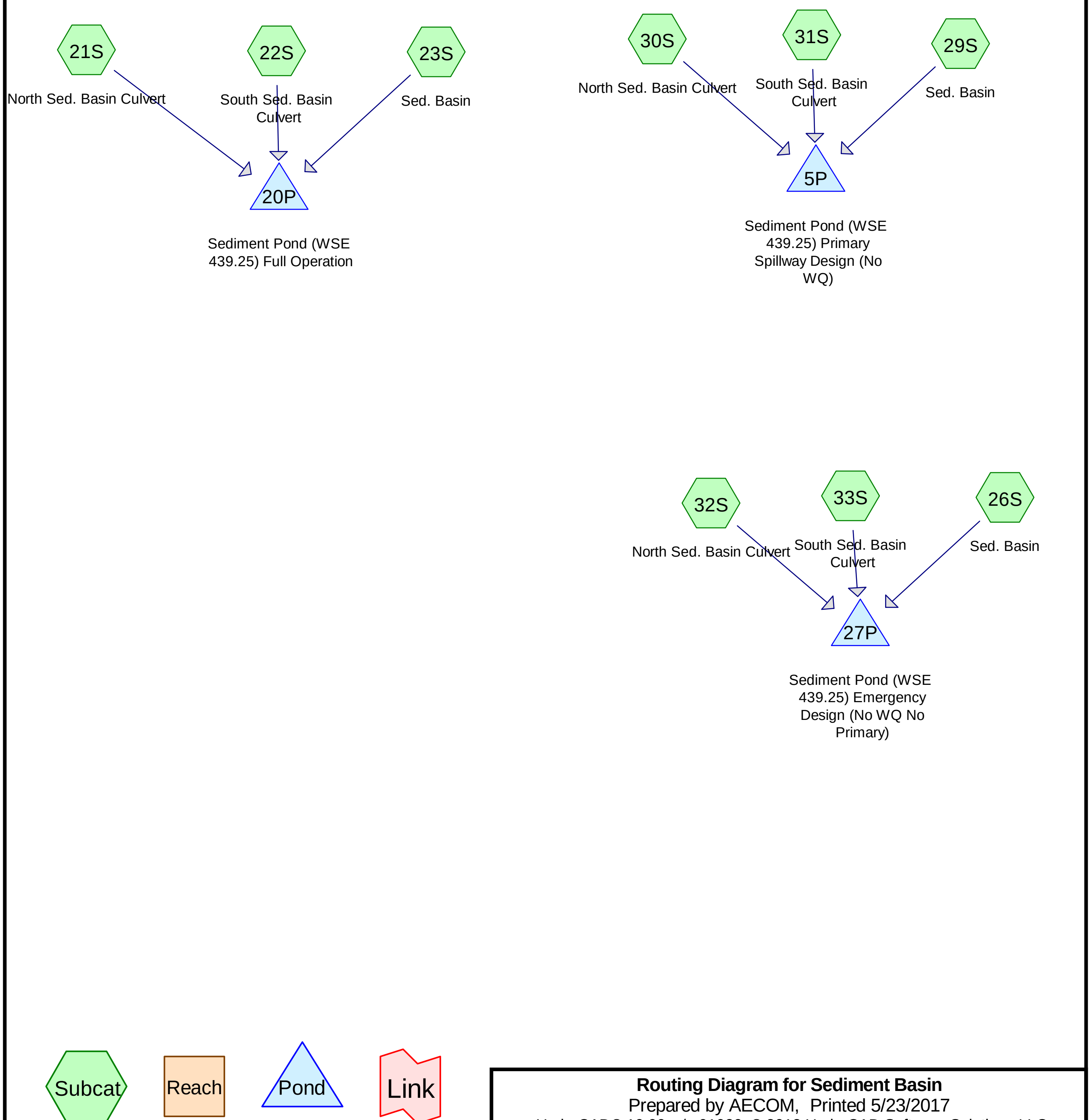
Elevation (feet)	Primary (cfs)	Elevation (feet)	Primary (cfs)
434.00	0.00	444.80	0.00
434.20	0.00	445.00	0.00
434.40	0.00	445.20	0.00
434.60	0.00	445.40	0.00
434.80	0.00	445.60	0.00
435.00	0.00	445.80	0.00
435.20	0.00	446.00	0.00
435.40	0.00	446.20	0.00
435.60	0.00	446.40	0.00
435.80	0.00	446.60	0.00
436.00	0.00	446.80	0.00
436.20	0.00	447.00	0.00
436.40	0.00	447.20	0.00
436.60	0.00	447.40	0.00
436.80	0.00	447.60	0.00
437.00	0.00	447.80	0.00
437.20	0.00	448.00	0.00
437.40	0.00	448.20	0.00
437.60	0.00	448.40	0.00
437.80	0.00	448.60	0.00
438.00	0.00	448.80	0.00
438.20	0.00	449.00	0.00
438.40	0.00	449.20	0.00
438.60	0.00	449.40	0.00
438.80	0.00	449.60	0.00
439.00	0.00	449.80	0.00
439.20	0.00	450.00	0.00
439.40	0.00	450.20	0.00
439.60	0.00	450.40	3.89
439.80	0.00	450.60	13.95
440.00	0.00	450.80	27.53
440.20	0.00	451.00	43.11
440.40	0.00	451.20	60.94
440.60	0.00	451.40	**81.32**
440.80	0.00		
441.00	0.00		
441.20	0.00		
441.40	0.00		
441.60	0.00		
441.80	0.00		
442.00	0.00		
442.20	0.00		
442.40	0.00		
442.60	0.00		
442.80	0.00		
443.00	0.00		
443.20	0.00		
443.40	0.00		
443.60	0.00		
443.80	0.00		
444.00	0.00		
444.20	0.00		
444.40	0.00		
444.60	0.00		

Stage-Area-Storage for Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)

Elevation (feet)	Surface (acres)	Storage (acre-feet)	Elevation (feet)	Surface (acres)	Storage (acre-feet)
434.00	0.030	0.000	444.80	1.040	5.715
434.20	0.050	0.008	445.00	1.060	5.925
434.40	0.070	0.020	445.20	1.078	6.139
434.60	0.090	0.036	445.40	1.096	6.356
434.80	0.110	0.056	445.60	1.114	6.577
435.00	0.130	0.080	445.80	1.132	6.802
435.20	0.148	0.108	446.00	1.150	7.030
435.40	0.166	0.139	446.20	1.168	7.262
435.60	0.184	0.174	446.40	1.186	7.497
435.80	0.202	0.213	446.60	1.204	7.736
436.00	0.220	0.255	446.80	1.222	7.979
436.20	0.236	0.301	447.00	1.240	8.225
436.40	0.252	0.349	447.20	1.258	8.475
436.60	0.268	0.401	447.40	1.276	8.728
436.80	0.284	0.457	447.60	1.294	8.985
437.00	0.300	0.515	447.80	1.312	9.246
437.20	0.318	0.577	448.00	1.330	9.510
437.40	0.336	0.642	448.20	1.346	9.778
437.60	0.354	0.711	448.40	1.362	10.048
437.80	0.372	0.784	448.60	1.378	10.322
438.00	0.390	0.860	448.80	1.394	10.600
438.20	0.410	0.940	449.00	1.410	10.880
438.40	0.430	1.024	449.20	1.428	11.164
438.60	0.450	1.112	449.40	1.446	11.451
438.80	0.470	1.204	449.60	1.464	11.742
439.00	0.490	1.300	449.80	1.482	12.037
439.20	0.508	1.400	450.00	1.500	12.335
439.40	0.526	1.503	450.20	1.516	12.637
439.60	0.544	1.610	450.40	1.532	12.941
439.80	0.562	1.721	450.60	1.548	13.249
440.00	0.580	1.835	450.80	1.564	13.561
440.20	0.598	1.953	451.00	1.580	13.875
440.40	0.616	2.074	451.20	1.616	14.195
440.60	0.634	2.199	451.40	**1.652**	**14.521**
440.80	0.652	2.328			
441.00	0.670	2.460			
441.20	0.690	2.596			
441.40	0.710	2.736			
441.60	0.730	2.880			
441.80	0.750	3.028			
442.00	0.770	3.180			
442.20	0.790	3.336			
442.40	0.810	3.496			
442.60	0.830	3.660			
442.80	0.850	3.828			
443.00	0.870	4.000			
443.20	0.888	4.176			
443.40	0.906	4.355			
443.60	0.924	4.538			
443.80	0.942	4.725			
444.00	0.960	4.915			
444.20	0.980	5.109			
444.40	1.000	5.307			
444.60	1.020	5.509			

ATTACHMENT C

HydroCAD report for Sediment Basin - 24-hr, 100-yr storm



Routing Diagram for Sediment Basin
Prepared by AECOM, Printed 5/23/2017
HydroCAD® 10.00 s/n 01089 © 2013 HydroCAD Software Solutions LLC

Summary for Subcatchment 21S: North Sed. Basin Culvert

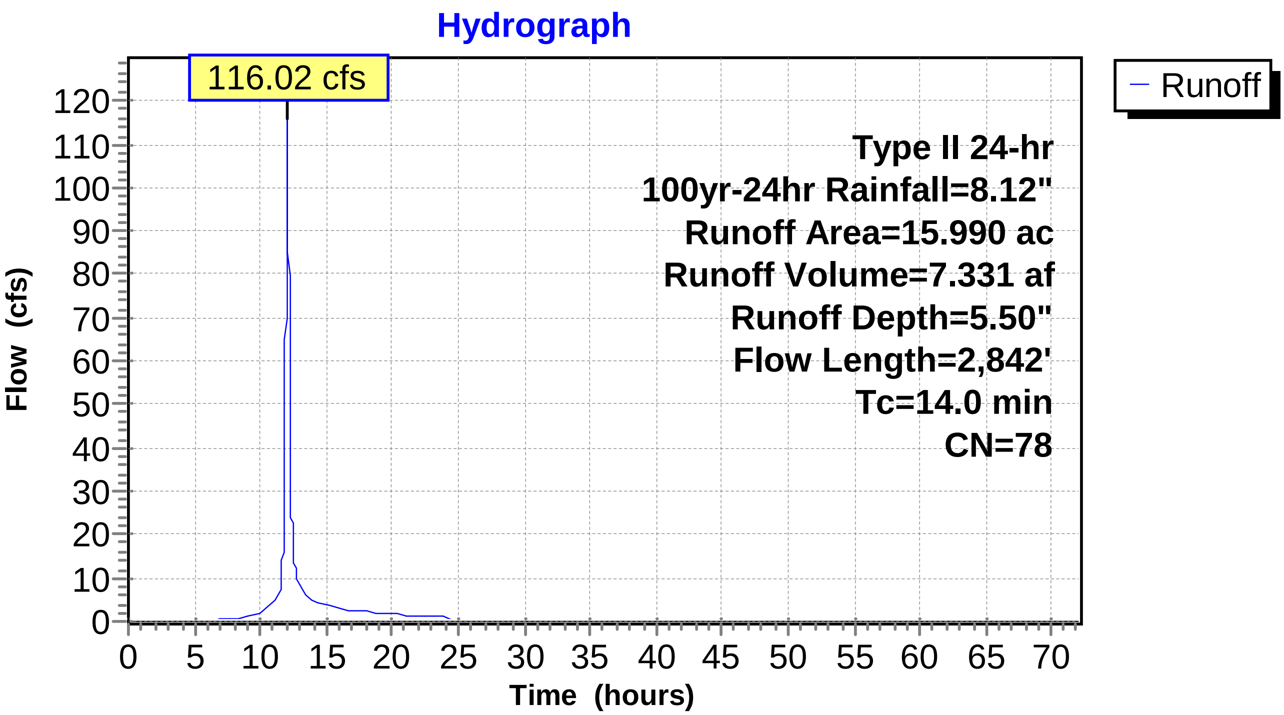
Runoff = 116.02 cfs @ 12.06 hrs, Volume= 7.331 af, Depth= 5.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 21S: North Sed. Basin Culvert



Hydrograph for Subcatchment 21S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.00	0.00
5.00	0.51	0.00	0.00
6.00	0.65	0.00	0.09
7.00	0.80	0.02	0.34
8.00	0.97	0.05	0.62
9.00	1.19	0.11	1.21
10.00	1.47	0.22	1.95
11.00	1.91	0.43	4.27
12.00	5.38	3.04	**102.67**
13.00	6.27	3.82	**8.42**
14.00	6.66	4.17	4.86
15.00	6.93	4.41	3.73
16.00	7.15	4.61	2.92
17.00	7.32	4.77	2.52
18.00	7.48	4.91	2.22
19.00	7.61	5.04	1.93
20.00	7.73	5.14	1.63
21.00	7.83	5.24	1.52
22.00	7.93	5.33	1.46
23.00	8.03	5.42	1.41
24.00	**8.12**	**5.50**	1.35
25.00	8.12	5.50	0.00
26.00	8.12	5.50	0.00
27.00	8.12	5.50	0.00
28.00	8.12	5.50	0.00
29.00	8.12	5.50	0.00
30.00	8.12	5.50	0.00
31.00	8.12	5.50	0.00
32.00	8.12	5.50	0.00
33.00	8.12	5.50	0.00
34.00	8.12	5.50	0.00
35.00	8.12	5.50	0.00
36.00	8.12	5.50	0.00
37.00	8.12	5.50	0.00
38.00	8.12	5.50	0.00
39.00	8.12	5.50	0.00
40.00	8.12	5.50	0.00
41.00	8.12	5.50	0.00
42.00	8.12	5.50	0.00
43.00	8.12	5.50	0.00
44.00	8.12	5.50	0.00
45.00	8.12	5.50	0.00
46.00	8.12	5.50	0.00
47.00	8.12	5.50	0.00
48.00	8.12	5.50	0.00
49.00	8.12	5.50	0.00
50.00	8.12	5.50	0.00
51.00	8.12	5.50	0.00
52.00	8.12	5.50	0.00
53.00	8.12	5.50	0.00
54.00	8.12	5.50	0.00
55.00	8.12	5.50	0.00
56.00	8.12	5.50	0.00
57.00	8.12	5.50	0.00
58.00	8.12	5.50	0.00
59.00	8.12	5.50	0.00
60.00	8.12	5.50	0.00
61.00	8.12	5.50	0.00
62.00	8.12	5.50	0.00
63.00	8.12	5.50	0.00
64.00	8.12	5.50	0.00
65.00	8.12	5.50	0.00
66.00	8.12	5.50	0.00
67.00	8.12	5.50	0.00
68.00	8.12	5.50	0.00
69.00	8.12	5.50	0.00
70.00	8.12	5.50	0.00
71.00	8.12	5.50	0.00
72.00	8.12	5.50	0.00

Summary for Subcatchment 22S: South Sed. Basin Culvert

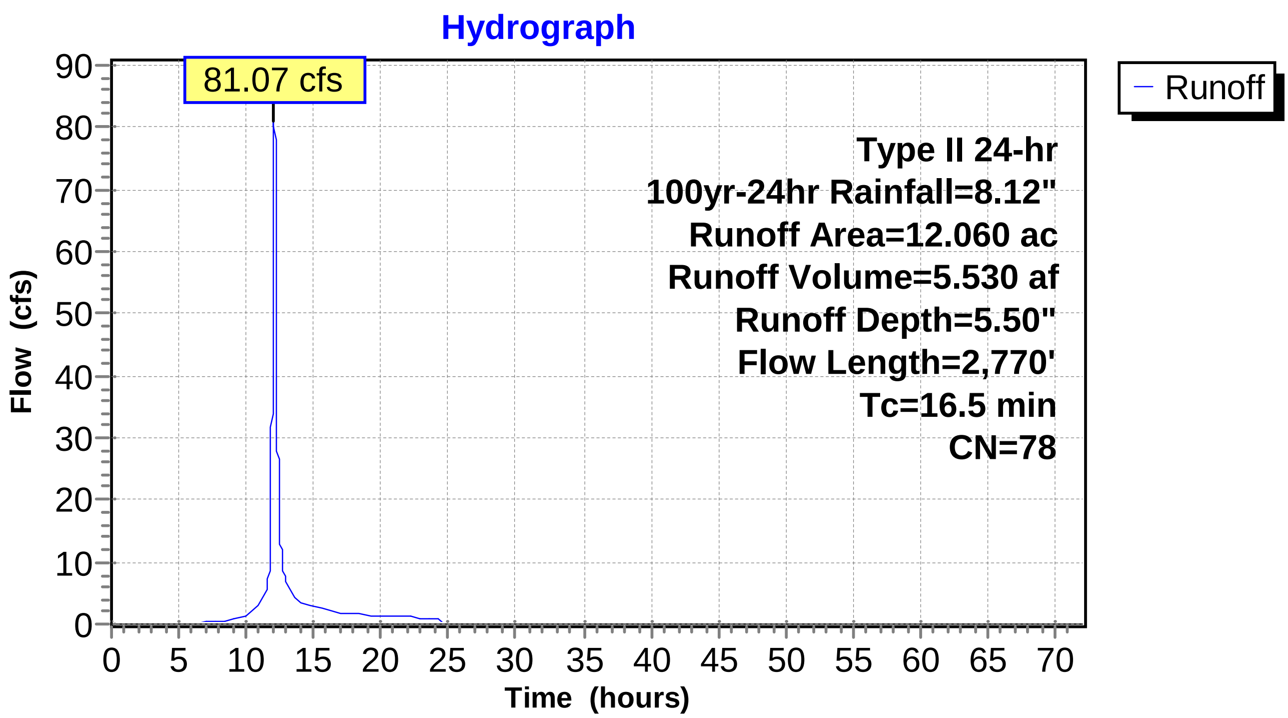
Runoff = 81.07 cfs @ 12.08 hrs, Volume= 5.530 af, Depth= 5.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 22S: South Sed. Basin Culvert



Hydrograph for Subcatchment 22S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.00	0.00
5.00	0.51	0.00	0.00
6.00	0.65	0.00	0.06
7.00	0.80	0.02	0.25
8.00	0.97	0.05	0.46
9.00	1.19	0.11	0.89
10.00	1.47	0.22	1.43
11.00	1.91	0.43	3.12
12.00	5.38	3.04	**65.37**
13.00	6.27	3.82	**6.58**
14.00	6.66	4.17	3.74
15.00	6.93	4.41	2.84
16.00	7.15	4.61	2.23
17.00	7.32	4.77	1.91
18.00	7.48	4.91	1.68
19.00	7.61	5.04	1.46
20.00	7.73	5.14	1.24
21.00	7.83	5.24	1.15
22.00	7.93	5.33	1.10
23.00	8.03	5.42	1.06
24.00	**8.12**	**5.50**	1.02
25.00	8.12	5.50	0.00
26.00	8.12	5.50	0.00
27.00	8.12	5.50	0.00
28.00	8.12	5.50	0.00
29.00	8.12	5.50	0.00
30.00	8.12	5.50	0.00
31.00	8.12	5.50	0.00
32.00	8.12	5.50	0.00
33.00	8.12	5.50	0.00
34.00	8.12	5.50	0.00
35.00	8.12	5.50	0.00
36.00	8.12	5.50	0.00
37.00	8.12	5.50	0.00
38.00	8.12	5.50	0.00
39.00	8.12	5.50	0.00
40.00	8.12	5.50	0.00
41.00	8.12	5.50	0.00
42.00	8.12	5.50	0.00
43.00	8.12	5.50	0.00
44.00	8.12	5.50	0.00
45.00	8.12	5.50	0.00
46.00	8.12	5.50	0.00
47.00	8.12	5.50	0.00
48.00	8.12	5.50	0.00
49.00	8.12	5.50	0.00
50.00	8.12	5.50	0.00
51.00	8.12	5.50	0.00
52.00	8.12	5.50	0.00
53.00	8.12	5.50	0.00
54.00	8.12	5.50	0.00
55.00	8.12	5.50	0.00
56.00	8.12	5.50	0.00
57.00	8.12	5.50	0.00
58.00	8.12	5.50	0.00
59.00	8.12	5.50	0.00
60.00	8.12	5.50	0.00
61.00	8.12	5.50	0.00
62.00	8.12	5.50	0.00
63.00	8.12	5.50	0.00
64.00	8.12	5.50	0.00
65.00	8.12	5.50	0.00
66.00	8.12	5.50	0.00
67.00	8.12	5.50	0.00
68.00	8.12	5.50	0.00
69.00	8.12	5.50	0.00
70.00	8.12	5.50	0.00
71.00	8.12	5.50	0.00
72.00	8.12	5.50	0.00

Summary for Subcatchment 23S: Sed. Basin

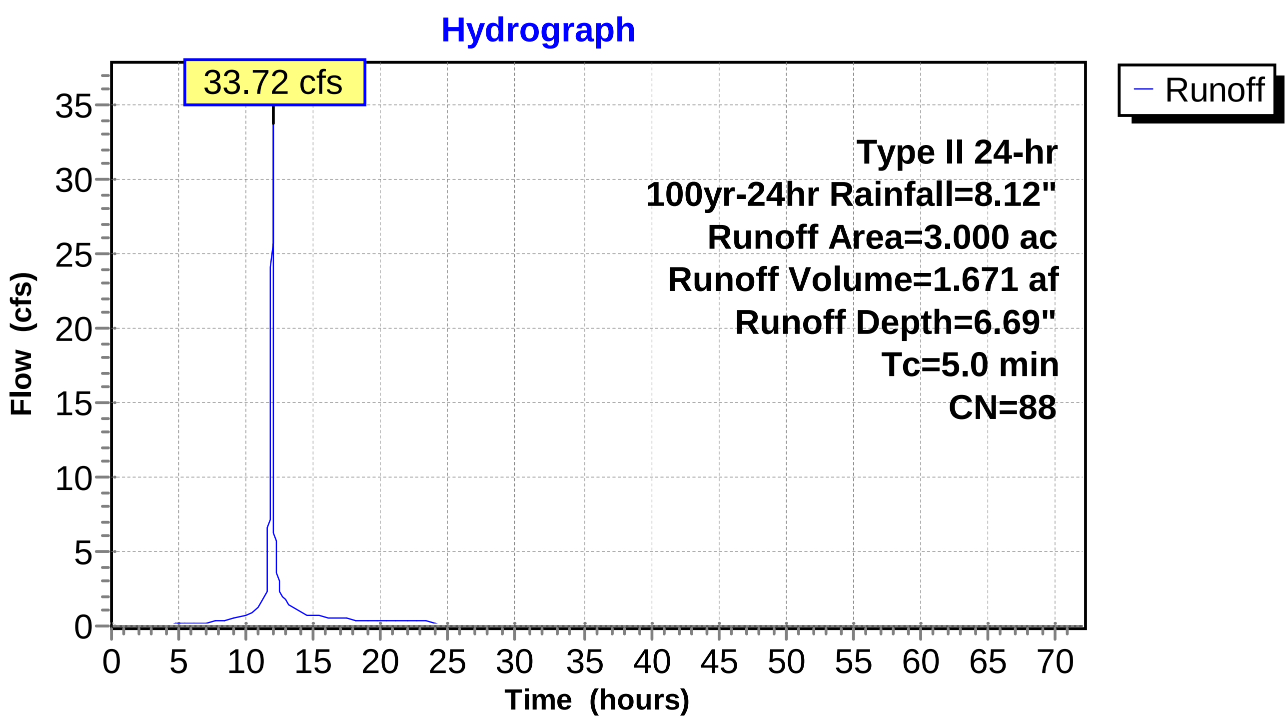
Runoff = 33.72 cfs @ 11.96 hrs, Volume= 1.671 af, Depth= 6.69"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	1.500	98	
*	1.500	78	
	3.000	88	Weighted Average
	1.500		50.00% Pervious Area
	1.500		50.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry, Min. Time of Concentration**

Subcatchment 23S: Sed. Basin



Hydrograph for Subcatchment 23S: Sed. Basin

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.01	0.05
5.00	0.51	0.04	0.10
6.00	0.65	0.08	0.17
7.00	0.80	0.15	0.23
8.00	0.97	0.24	0.30
9.00	1.19	0.37	0.49
10.00	1.47	0.56	0.68
11.00	1.91	0.89	**1.33**
12.00	5.38	4.03	**28.04**
13.00	6.27	4.88	1.54
14.00	6.66	5.26	0.92
15.00	6.93	5.53	0.73
16.00	7.15	5.73	0.56
17.00	7.32	5.91	0.49
18.00	7.48	6.06	0.44
19.00	7.61	6.19	0.38
20.00	7.73	6.30	0.32
21.00	7.83	6.41	0.30
22.00	7.93	6.50	0.29
23.00	8.03	6.60	0.28
24.00	**8.12**	**6.69**	0.27
25.00	8.12	6.69	0.00
26.00	8.12	6.69	0.00
27.00	8.12	6.69	0.00
28.00	8.12	6.69	0.00
29.00	8.12	6.69	0.00
30.00	8.12	6.69	0.00
31.00	8.12	6.69	0.00
32.00	8.12	6.69	0.00
33.00	8.12	6.69	0.00
34.00	8.12	6.69	0.00
35.00	8.12	6.69	0.00
36.00	8.12	6.69	0.00
37.00	8.12	6.69	0.00
38.00	8.12	6.69	0.00
39.00	8.12	6.69	0.00
40.00	8.12	6.69	0.00
41.00	8.12	6.69	0.00
42.00	8.12	6.69	0.00
43.00	8.12	6.69	0.00
44.00	8.12	6.69	0.00
45.00	8.12	6.69	0.00
46.00	8.12	6.69	0.00
47.00	8.12	6.69	0.00
48.00	8.12	6.69	0.00
49.00	8.12	6.69	0.00
50.00	8.12	6.69	0.00
51.00	8.12	6.69	0.00
52.00	8.12	6.69	0.00
53.00	8.12	6.69	0.00

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
54.00	8.12	6.69	0.00
55.00	8.12	6.69	0.00
56.00	8.12	6.69	0.00
57.00	8.12	6.69	0.00
58.00	8.12	6.69	0.00
59.00	8.12	6.69	0.00
60.00	8.12	6.69	0.00
61.00	8.12	6.69	0.00
62.00	8.12	6.69	0.00
63.00	8.12	6.69	0.00
64.00	8.12	6.69	0.00
65.00	8.12	6.69	0.00
66.00	8.12	6.69	0.00
67.00	8.12	6.69	0.00
68.00	8.12	6.69	0.00
69.00	8.12	6.69	0.00
70.00	8.12	6.69	0.00
71.00	8.12	6.69	0.00
72.00	8.12	6.69	0.00

Summary for Subcatchment 26S: Sed. Basin

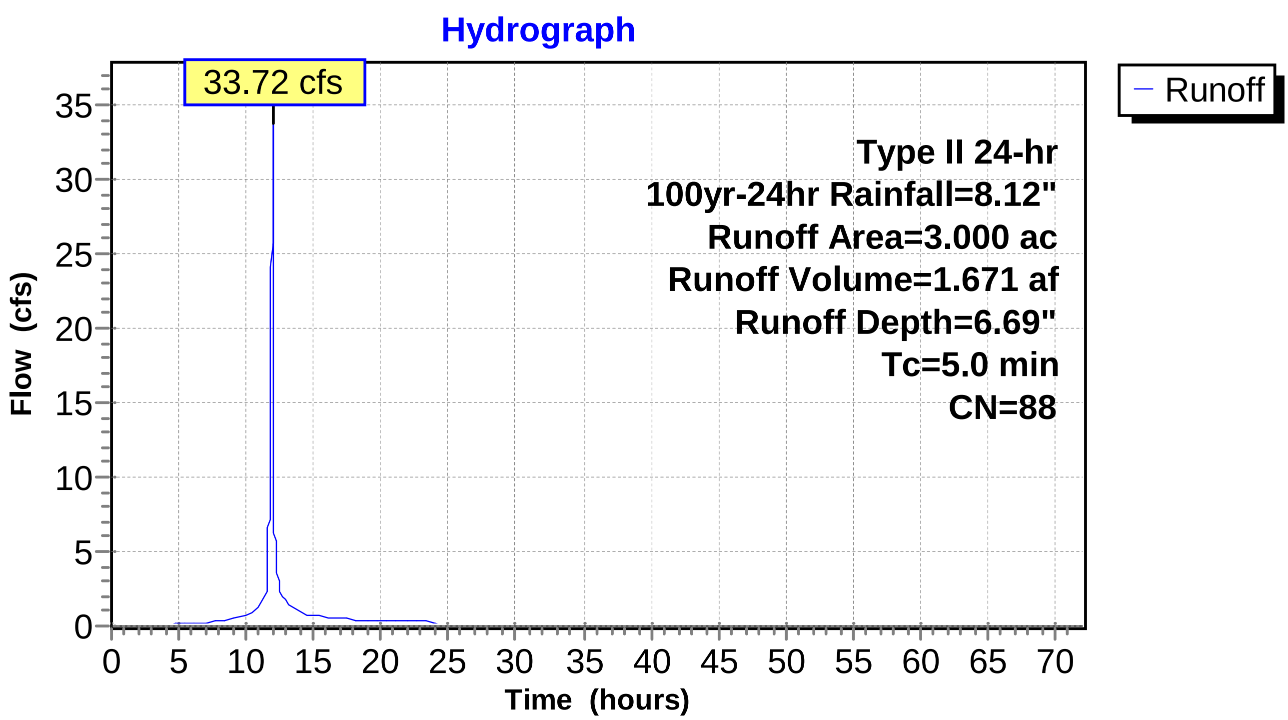
Runoff = 33.72 cfs @ 11.96 hrs, Volume= 1.671 af, Depth= 6.69"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	1.500	98	
*	1.500	78	
	3.000	88	Weighted Average
	1.500		50.00% Pervious Area
	1.500		50.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry, Min. Time of Concentration**

Subcatchment 26S: Sed. Basin



Hydrograph for Subcatchment 26S: Sed. Basin

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.01	0.05
5.00	0.51	0.04	0.10
6.00	0.65	0.08	0.17
7.00	0.80	0.15	0.23
8.00	0.97	0.24	0.30
9.00	1.19	0.37	0.49
10.00	1.47	0.56	0.68
11.00	1.91	0.89	**1.33**
12.00	5.38	4.03	**28.04**
13.00	6.27	4.88	1.54
14.00	6.66	5.26	0.92
15.00	6.93	5.53	0.73
16.00	7.15	5.73	0.56
17.00	7.32	5.91	0.49
18.00	7.48	6.06	0.44
19.00	7.61	6.19	0.38
20.00	7.73	6.30	0.32
21.00	7.83	6.41	0.30
22.00	7.93	6.50	0.29
23.00	8.03	6.60	0.28
24.00	**8.12**	**6.69**	0.27
25.00	8.12	6.69	0.00
26.00	8.12	6.69	0.00
27.00	8.12	6.69	0.00
28.00	8.12	6.69	0.00
29.00	8.12	6.69	0.00
30.00	8.12	6.69	0.00
31.00	8.12	6.69	0.00
32.00	8.12	6.69	0.00
33.00	8.12	6.69	0.00
34.00	8.12	6.69	0.00
35.00	8.12	6.69	0.00
36.00	8.12	6.69	0.00
37.00	8.12	6.69	0.00
38.00	8.12	6.69	0.00
39.00	8.12	6.69	0.00
40.00	8.12	6.69	0.00
41.00	8.12	6.69	0.00
42.00	8.12	6.69	0.00
43.00	8.12	6.69	0.00
44.00	8.12	6.69	0.00
45.00	8.12	6.69	0.00
46.00	8.12	6.69	0.00
47.00	8.12	6.69	0.00
48.00	8.12	6.69	0.00
49.00	8.12	6.69	0.00
50.00	8.12	6.69	0.00
51.00	8.12	6.69	0.00
52.00	8.12	6.69	0.00
53.00	8.12	6.69	0.00

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
54.00	8.12	6.69	0.00
55.00	8.12	6.69	0.00
56.00	8.12	6.69	0.00
57.00	8.12	6.69	0.00
58.00	8.12	6.69	0.00
59.00	8.12	6.69	0.00
60.00	8.12	6.69	0.00
61.00	8.12	6.69	0.00
62.00	8.12	6.69	0.00
63.00	8.12	6.69	0.00
64.00	8.12	6.69	0.00
65.00	8.12	6.69	0.00
66.00	8.12	6.69	0.00
67.00	8.12	6.69	0.00
68.00	8.12	6.69	0.00
69.00	8.12	6.69	0.00
70.00	8.12	6.69	0.00
71.00	8.12	6.69	0.00
72.00	8.12	6.69	0.00

Summary for Subcatchment 29S: Sed. Basin

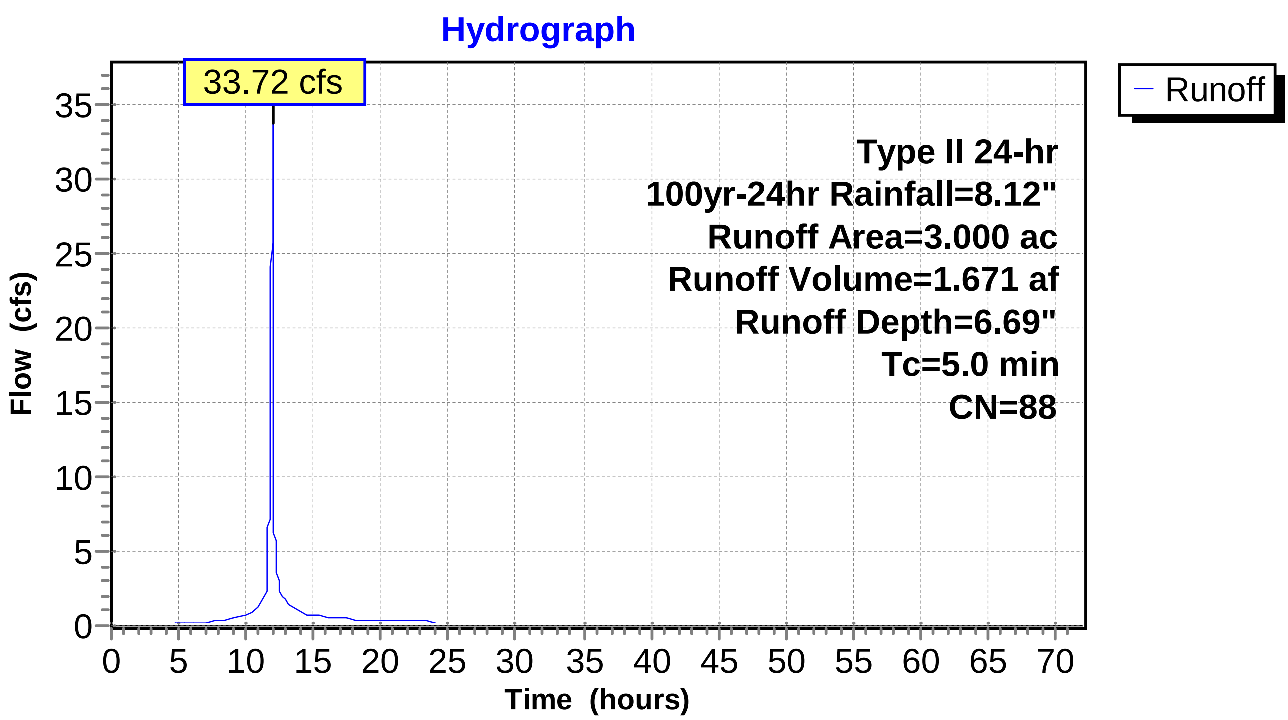
Runoff = 33.72 cfs @ 11.96 hrs, Volume= 1.671 af, Depth= 6.69"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	1.500	98	
*	1.500	78	
	3.000	88	Weighted Average
	1.500		50.00% Pervious Area
	1.500		50.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry, Min. Time of Concentration**

Subcatchment 29S: Sed. Basin



Hydrograph for Subcatchment 29S: Sed. Basin

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.01	0.05
5.00	0.51	0.04	0.10
6.00	0.65	0.08	0.17
7.00	0.80	0.15	0.23
8.00	0.97	0.24	0.30
9.00	1.19	0.37	0.49
10.00	1.47	0.56	0.68
11.00	1.91	0.89	**1.33**
12.00	5.38	4.03	**28.04**
13.00	6.27	4.88	1.54
14.00	6.66	5.26	0.92
15.00	6.93	5.53	0.73
16.00	7.15	5.73	0.56
17.00	7.32	5.91	0.49
18.00	7.48	6.06	0.44
19.00	7.61	6.19	0.38
20.00	7.73	6.30	0.32
21.00	7.83	6.41	0.30
22.00	7.93	6.50	0.29
23.00	8.03	6.60	0.28
24.00	**8.12**	**6.69**	0.27
25.00	8.12	6.69	0.00
26.00	8.12	6.69	0.00
27.00	8.12	6.69	0.00
28.00	8.12	6.69	0.00
29.00	8.12	6.69	0.00
30.00	8.12	6.69	0.00
31.00	8.12	6.69	0.00
32.00	8.12	6.69	0.00
33.00	8.12	6.69	0.00
34.00	8.12	6.69	0.00
35.00	8.12	6.69	0.00
36.00	8.12	6.69	0.00
37.00	8.12	6.69	0.00
38.00	8.12	6.69	0.00
39.00	8.12	6.69	0.00
40.00	8.12	6.69	0.00
41.00	8.12	6.69	0.00
42.00	8.12	6.69	0.00
43.00	8.12	6.69	0.00
44.00	8.12	6.69	0.00
45.00	8.12	6.69	0.00
46.00	8.12	6.69	0.00
47.00	8.12	6.69	0.00
48.00	8.12	6.69	0.00
49.00	8.12	6.69	0.00
50.00	8.12	6.69	0.00
51.00	8.12	6.69	0.00
52.00	8.12	6.69	0.00
53.00	8.12	6.69	0.00
54.00	8.12	6.69	0.00
55.00	8.12	6.69	0.00
56.00	8.12	6.69	0.00
57.00	8.12	6.69	0.00
58.00	8.12	6.69	0.00
59.00	8.12	6.69	0.00
60.00	8.12	6.69	0.00
61.00	8.12	6.69	0.00
62.00	8.12	6.69	0.00
63.00	8.12	6.69	0.00
64.00	8.12	6.69	0.00
65.00	8.12	6.69	0.00
66.00	8.12	6.69	0.00
67.00	8.12	6.69	0.00
68.00	8.12	6.69	0.00
69.00	8.12	6.69	0.00
70.00	8.12	6.69	0.00
71.00	8.12	6.69	0.00
72.00	8.12	6.69	0.00

Summary for Subcatchment 30S: North Sed. Basin Culvert

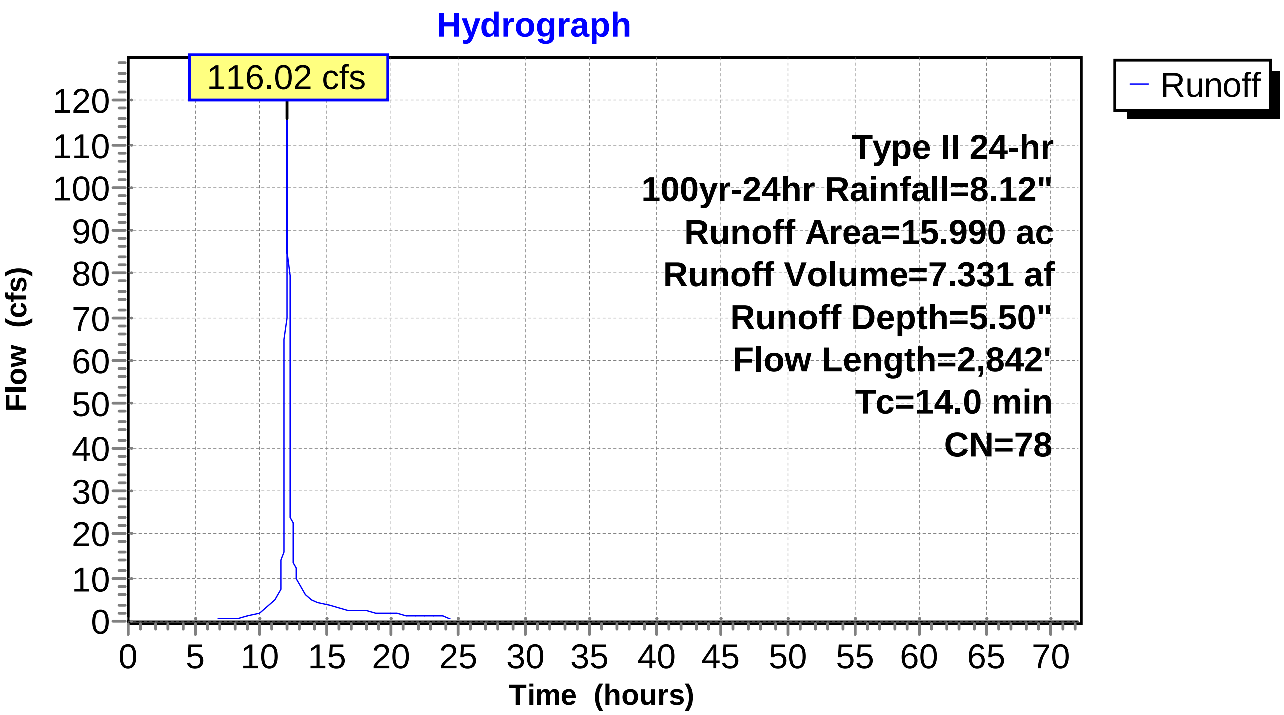
Runoff = 116.02 cfs @ 12.06 hrs, Volume= 7.331 af, Depth= 5.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow**
					Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow**
					Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace**
					Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down**
					Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1**
					Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2**
					Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3**
					Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4**
					Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 30S: North Sed. Basin Culvert



Hydrograph for Subcatchment 30S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.00	0.00
5.00	0.51	0.00	0.00
6.00	0.65	0.00	0.09
7.00	0.80	0.02	0.34
8.00	0.97	0.05	0.62
9.00	1.19	0.11	1.21
10.00	1.47	0.22	1.95
11.00	1.91	0.43	4.27
12.00	5.38	3.04	**102.67**
13.00	6.27	3.82	**8.42**
14.00	6.66	4.17	4.86
15.00	6.93	4.41	3.73
16.00	7.15	4.61	2.92
17.00	7.32	4.77	2.52
18.00	7.48	4.91	2.22
19.00	7.61	5.04	1.93
20.00	7.73	5.14	1.63
21.00	7.83	5.24	1.52
22.00	7.93	5.33	1.46
23.00	8.03	5.42	1.41
24.00	**8.12**	**5.50**	1.35
25.00	8.12	5.50	0.00
26.00	8.12	5.50	0.00
27.00	8.12	5.50	0.00
28.00	8.12	5.50	0.00
29.00	8.12	5.50	0.00
30.00	8.12	5.50	0.00
31.00	8.12	5.50	0.00
32.00	8.12	5.50	0.00
33.00	8.12	5.50	0.00
34.00	8.12	5.50	0.00
35.00	8.12	5.50	0.00
36.00	8.12	5.50	0.00
37.00	8.12	5.50	0.00
38.00	8.12	5.50	0.00
39.00	8.12	5.50	0.00
40.00	8.12	5.50	0.00
41.00	8.12	5.50	0.00
42.00	8.12	5.50	0.00
43.00	8.12	5.50	0.00
44.00	8.12	5.50	0.00
45.00	8.12	5.50	0.00
46.00	8.12	5.50	0.00
47.00	8.12	5.50	0.00
48.00	8.12	5.50	0.00
49.00	8.12	5.50	0.00
50.00	8.12	5.50	0.00
51.00	8.12	5.50	0.00
52.00	8.12	5.50	0.00
53.00	8.12	5.50	0.00

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
54.00	8.12	5.50	0.00
55.00	8.12	5.50	0.00
56.00	8.12	5.50	0.00
57.00	8.12	5.50	0.00
58.00	8.12	5.50	0.00
59.00	8.12	5.50	0.00
60.00	8.12	5.50	0.00
61.00	8.12	5.50	0.00
62.00	8.12	5.50	0.00
63.00	8.12	5.50	0.00
64.00	8.12	5.50	0.00
65.00	8.12	5.50	0.00
66.00	8.12	5.50	0.00
67.00	8.12	5.50	0.00
68.00	8.12	5.50	0.00
69.00	8.12	5.50	0.00
70.00	8.12	5.50	0.00
71.00	8.12	5.50	0.00
72.00	8.12	5.50	0.00

Summary for Subcatchment 31S: South Sed. Basin Culvert

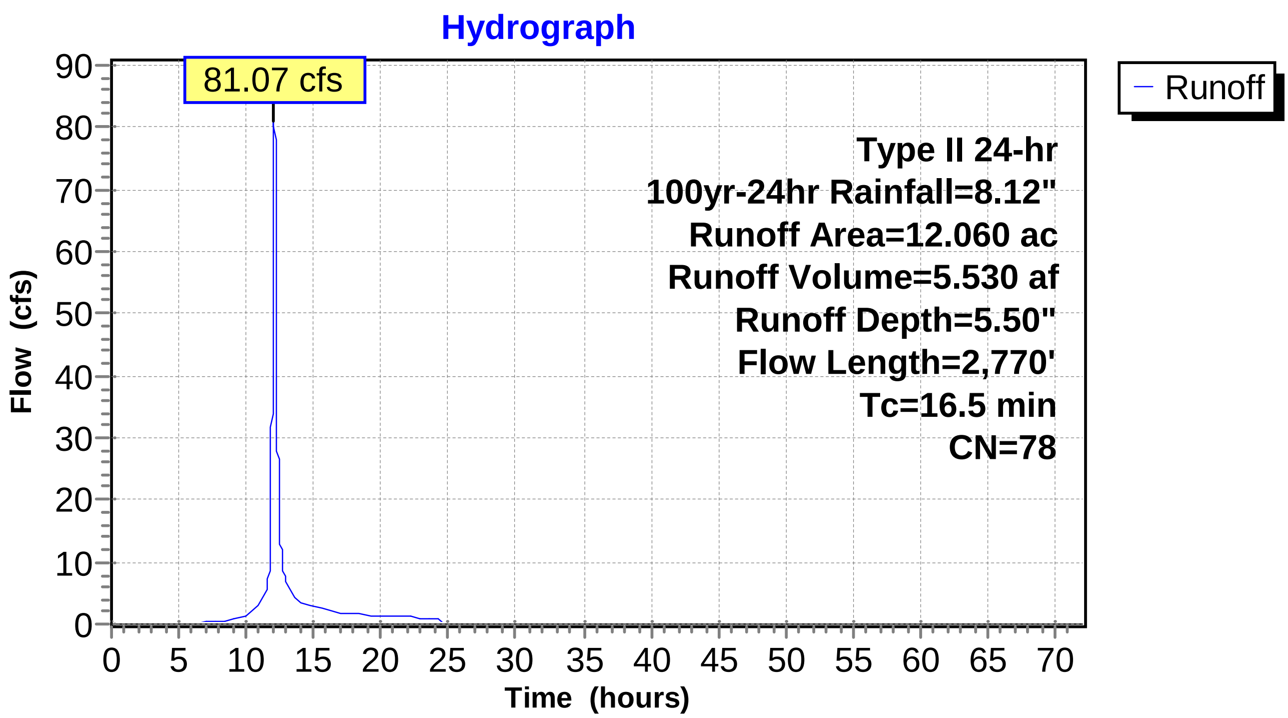
Runoff = 81.07 cfs @ 12.08 hrs, Volume= 5.530 af, Depth= 5.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 31S: South Sed. Basin Culvert



Hydrograph for Subcatchment 31S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.00	0.00
5.00	0.51	0.00	0.00
6.00	0.65	0.00	0.06
7.00	0.80	0.02	0.25
8.00	0.97	0.05	0.46
9.00	1.19	0.11	0.89
10.00	1.47	0.22	1.43
11.00	1.91	0.43	3.12
12.00	5.38	3.04	**65.37**
13.00	6.27	3.82	**6.58**
14.00	6.66	4.17	3.74
15.00	6.93	4.41	2.84
16.00	7.15	4.61	2.23
17.00	7.32	4.77	1.91
18.00	7.48	4.91	1.68
19.00	7.61	5.04	1.46
20.00	7.73	5.14	1.24
21.00	7.83	5.24	1.15
22.00	7.93	5.33	1.10
23.00	8.03	5.42	1.06
24.00	**8.12**	**5.50**	1.02
25.00	8.12	5.50	0.00
26.00	8.12	5.50	0.00
27.00	8.12	5.50	0.00
28.00	8.12	5.50	0.00
29.00	8.12	5.50	0.00
30.00	8.12	5.50	0.00
31.00	8.12	5.50	0.00
32.00	8.12	5.50	0.00
33.00	8.12	5.50	0.00
34.00	8.12	5.50	0.00
35.00	8.12	5.50	0.00
36.00	8.12	5.50	0.00
37.00	8.12	5.50	0.00
38.00	8.12	5.50	0.00
39.00	8.12	5.50	0.00
40.00	8.12	5.50	0.00
41.00	8.12	5.50	0.00
42.00	8.12	5.50	0.00
43.00	8.12	5.50	0.00
44.00	8.12	5.50	0.00
45.00	8.12	5.50	0.00
46.00	8.12	5.50	0.00
47.00	8.12	5.50	0.00
48.00	8.12	5.50	0.00
49.00	8.12	5.50	0.00
50.00	8.12	5.50	0.00
51.00	8.12	5.50	0.00
52.00	8.12	5.50	0.00
53.00	8.12	5.50	0.00
54.00	8.12	5.50	0.00
55.00	8.12	5.50	0.00
56.00	8.12	5.50	0.00
57.00	8.12	5.50	0.00
58.00	8.12	5.50	0.00
59.00	8.12	5.50	0.00
60.00	8.12	5.50	0.00
61.00	8.12	5.50	0.00
62.00	8.12	5.50	0.00
63.00	8.12	5.50	0.00
64.00	8.12	5.50	0.00
65.00	8.12	5.50	0.00
66.00	8.12	5.50	0.00
67.00	8.12	5.50	0.00
68.00	8.12	5.50	0.00
69.00	8.12	5.50	0.00
70.00	8.12	5.50	0.00
71.00	8.12	5.50	0.00
72.00	8.12	5.50	0.00

Summary for Subcatchment 32S: North Sed. Basin Culvert

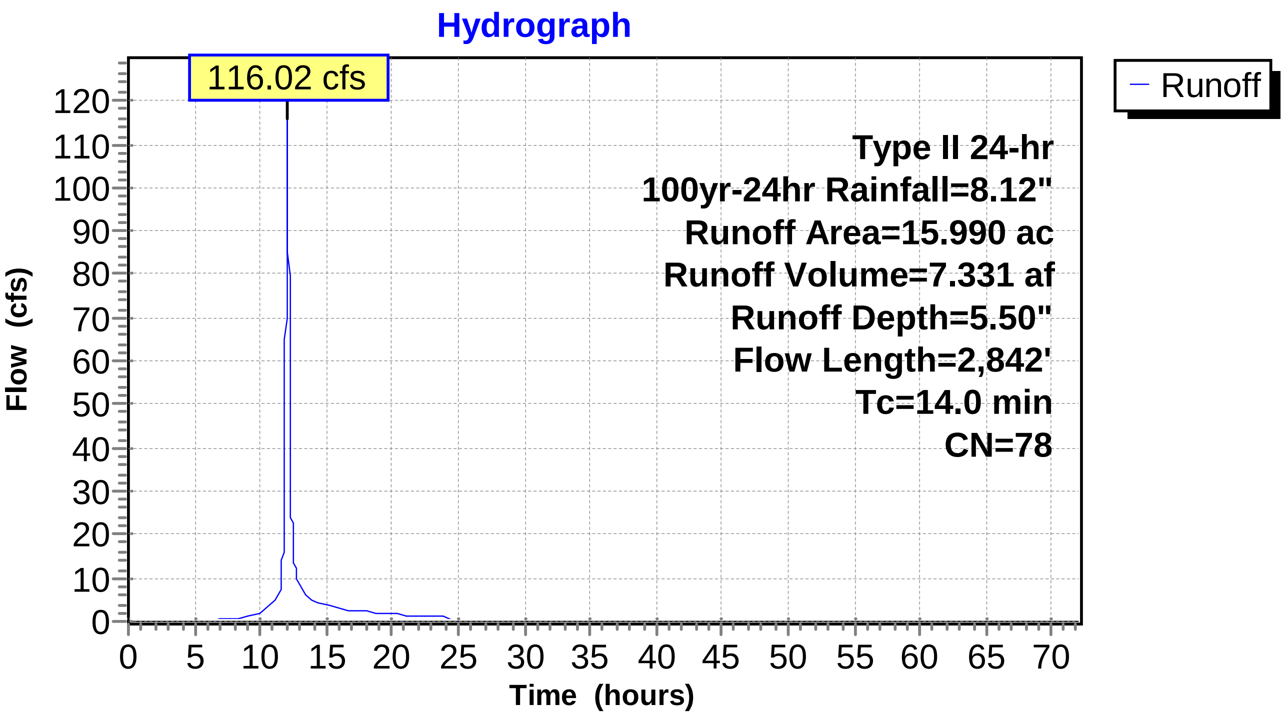
Runoff = 116.02 cfs @ 12.06 hrs, Volume= 7.331 af, Depth= 5.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 32S: North Sed. Basin Culvert



Hydrograph for Subcatchment 32S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.00	0.00
5.00	0.51	0.00	0.00
6.00	0.65	0.00	0.09
7.00	0.80	0.02	0.34
8.00	0.97	0.05	0.62
9.00	1.19	0.11	1.21
10.00	1.47	0.22	1.95
11.00	1.91	0.43	4.27
12.00	5.38	3.04	**102.67**
13.00	6.27	3.82	**8.42**
14.00	6.66	4.17	4.86
15.00	6.93	4.41	3.73
16.00	7.15	4.61	2.92
17.00	7.32	4.77	2.52
18.00	7.48	4.91	2.22
19.00	7.61	5.04	1.93
20.00	7.73	5.14	1.63
21.00	7.83	5.24	1.52
22.00	7.93	5.33	1.46
23.00	8.03	5.42	1.41
24.00	**8.12**	**5.50**	1.35
25.00	8.12	5.50	0.00
26.00	8.12	5.50	0.00
27.00	8.12	5.50	0.00
28.00	8.12	5.50	0.00
29.00	8.12	5.50	0.00
30.00	8.12	5.50	0.00
31.00	8.12	5.50	0.00
32.00	8.12	5.50	0.00
33.00	8.12	5.50	0.00
34.00	8.12	5.50	0.00
35.00	8.12	5.50	0.00
36.00	8.12	5.50	0.00
37.00	8.12	5.50	0.00
38.00	8.12	5.50	0.00
39.00	8.12	5.50	0.00
40.00	8.12	5.50	0.00
41.00	8.12	5.50	0.00
42.00	8.12	5.50	0.00
43.00	8.12	5.50	0.00
44.00	8.12	5.50	0.00
45.00	8.12	5.50	0.00
46.00	8.12	5.50	0.00
47.00	8.12	5.50	0.00
48.00	8.12	5.50	0.00
49.00	8.12	5.50	0.00
50.00	8.12	5.50	0.00
51.00	8.12	5.50	0.00
52.00	8.12	5.50	0.00
53.00	8.12	5.50	0.00
54.00	8.12	5.50	0.00
55.00	8.12	5.50	0.00
56.00	8.12	5.50	0.00
57.00	8.12	5.50	0.00
58.00	8.12	5.50	0.00
59.00	8.12	5.50	0.00
60.00	8.12	5.50	0.00
61.00	8.12	5.50	0.00
62.00	8.12	5.50	0.00
63.00	8.12	5.50	0.00
64.00	8.12	5.50	0.00
65.00	8.12	5.50	0.00
66.00	8.12	5.50	0.00
67.00	8.12	5.50	0.00
68.00	8.12	5.50	0.00
69.00	8.12	5.50	0.00
70.00	8.12	5.50	0.00
71.00	8.12	5.50	0.00
72.00	8.12	5.50	0.00

Summary for Subcatchment 33S: South Sed. Basin Culvert

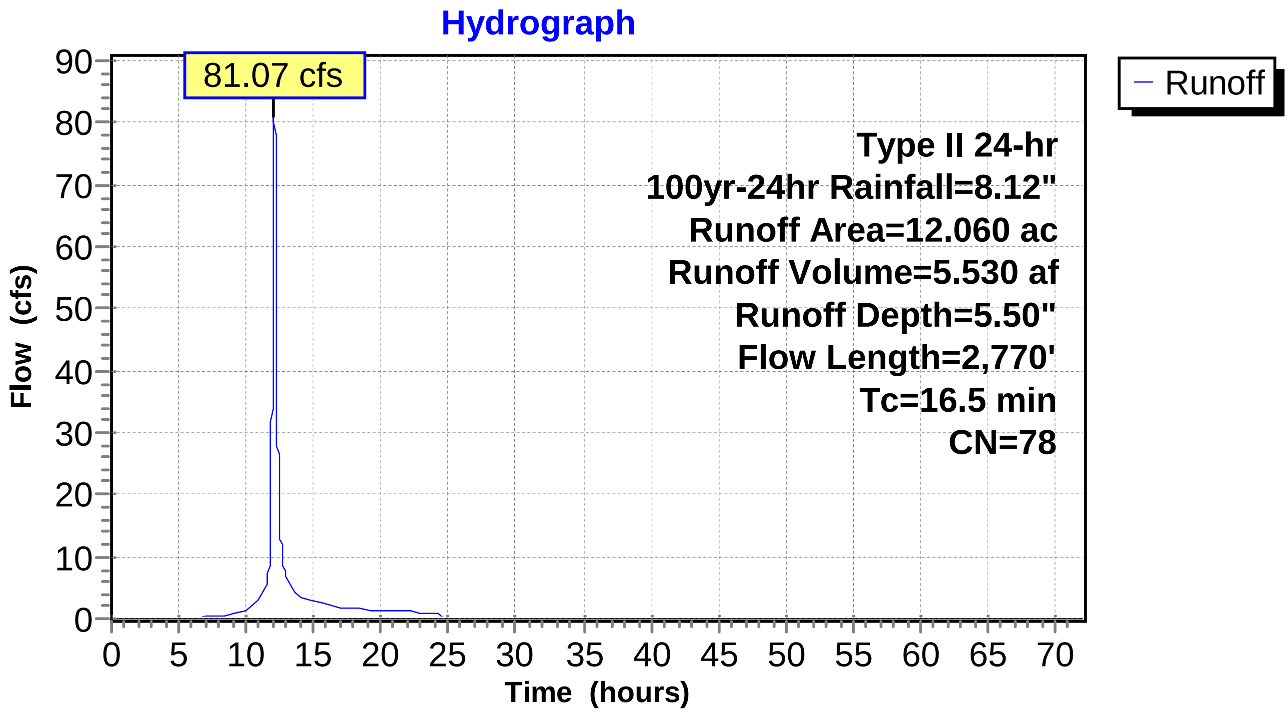
Runoff = 81.07 cfs @ 12.08 hrs, Volume= 5.530 af, Depth= 5.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 100yr-24hr Rainfall=8.12"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 33S: South Sed. Basin Culvert



Hydrograph for Subcatchment 33S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.09	0.00	0.00
2.00	0.18	0.00	0.00
3.00	0.28	0.00	0.00
4.00	0.39	0.00	0.00
5.00	0.51	0.00	0.00
6.00	0.65	0.00	0.06
7.00	0.80	0.02	0.25
8.00	0.97	0.05	0.46
9.00	1.19	0.11	0.89
10.00	1.47	0.22	1.43
11.00	1.91	0.43	3.12
12.00	5.38	3.04	**65.37**
13.00	6.27	3.82	**6.58**
14.00	6.66	4.17	3.74
15.00	6.93	4.41	2.84
16.00	7.15	4.61	2.23
17.00	7.32	4.77	1.91
18.00	7.48	4.91	1.68
19.00	7.61	5.04	1.46
20.00	7.73	5.14	1.24
21.00	7.83	5.24	1.15
22.00	7.93	5.33	1.10
23.00	8.03	5.42	1.06
24.00	**8.12**	**5.50**	1.02
25.00	8.12	5.50	0.00
26.00	8.12	5.50	0.00
27.00	8.12	5.50	0.00
28.00	8.12	5.50	0.00
29.00	8.12	5.50	0.00
30.00	8.12	5.50	0.00
31.00	8.12	5.50	0.00
32.00	8.12	5.50	0.00
33.00	8.12	5.50	0.00
34.00	8.12	5.50	0.00
35.00	8.12	5.50	0.00
36.00	8.12	5.50	0.00
37.00	8.12	5.50	0.00
38.00	8.12	5.50	0.00
39.00	8.12	5.50	0.00
40.00	8.12	5.50	0.00
41.00	8.12	5.50	0.00
42.00	8.12	5.50	0.00
43.00	8.12	5.50	0.00
44.00	8.12	5.50	0.00
45.00	8.12	5.50	0.00
46.00	8.12	5.50	0.00
47.00	8.12	5.50	0.00
48.00	8.12	5.50	0.00
49.00	8.12	5.50	0.00
50.00	8.12	5.50	0.00
51.00	8.12	5.50	0.00
52.00	8.12	5.50	0.00
53.00	8.12	5.50	0.00
54.00	8.12	5.50	0.00
55.00	8.12	5.50	0.00
56.00	8.12	5.50	0.00
57.00	8.12	5.50	0.00
58.00	8.12	5.50	0.00
59.00	8.12	5.50	0.00
60.00	8.12	5.50	0.00
61.00	8.12	5.50	0.00
62.00	8.12	5.50	0.00
63.00	8.12	5.50	0.00
64.00	8.12	5.50	0.00
65.00	8.12	5.50	0.00
66.00	8.12	5.50	0.00
67.00	8.12	5.50	0.00
68.00	8.12	5.50	0.00
69.00	8.12	5.50	0.00
70.00	8.12	5.50	0.00
71.00	8.12	5.50	0.00
72.00	8.12	5.50	0.00

Summary for Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)

Inflow Area = 31.050 ac, 4.83% Impervious, Inflow Depth = 5.62" for 100yr-24hr event
Inflow = 208.46 cfs @ 12.05 hrs, Volume= 14.532 af
Outflow = 6.57 cfs @ 15.46 hrs, Volume= 3.623 af, Atten= 97%, Lag= 204.6 min
Primary = 6.57 cfs @ 15.46 hrs, Volume= 3.623 af
Secondary = 0.01 cfs @ 15.46 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Starting Elev= 439.25' Surf.Area= 0.512 ac Storage= 1.425 af
Peak Elev= 450.25' @ 15.46 hrs Surf.Area= 1.520 ac Storage= 12.713 af (11.288 af above start)
Flood Elev= 450.50' Surf.Area= 1.540 ac Storage= 13.095 af (11.670 af above start)

Plug-Flow detention time= 616.5 min calculated for 2.197 af (15% of inflow)
Center-of-Mass det. time= 321.3 min (1,129.5 - 808.2)

Volume	Invert	Avail.Storage	Storage Description
#1	434.00'	14.687 af	**Sediment Basin (Prismatic)** Listed below (Recalc)

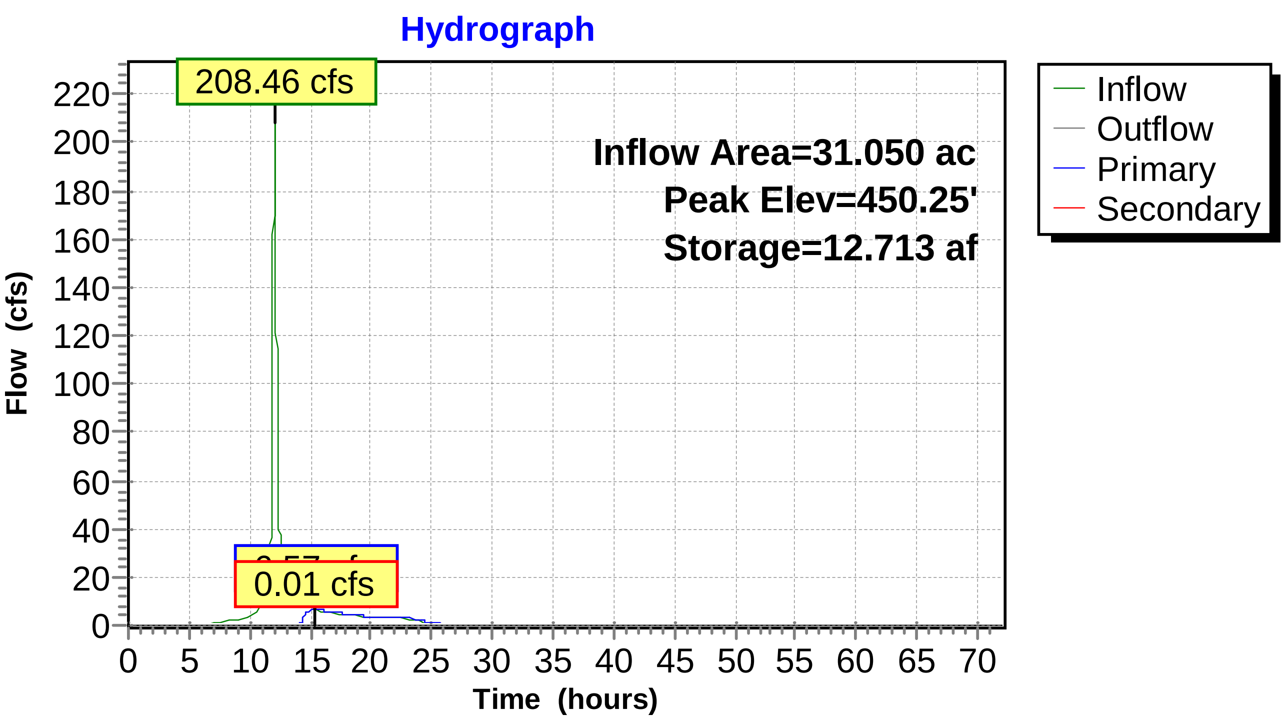
Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
434.00	0.030	0.000	0.000
435.00	0.130	0.080	0.080
436.00	0.220	0.175	0.255
437.00	0.300	0.260	0.515
438.00	0.390	0.345	0.860
439.00	0.490	0.440	1.300
440.00	0.580	0.535	1.835
441.00	0.670	0.625	2.460
442.00	0.770	0.720	3.180
443.00	0.870	0.820	4.000
444.00	0.960	0.915	4.915
445.00	1.060	1.010	5.925
446.00	1.150	1.105	7.030
447.00	1.240	1.195	8.225
448.00	1.330	1.285	9.510
449.00	1.410	1.370	10.880
450.00	1.500	1.455	12.335
451.00	1.580	1.540	13.875
451.50	1.670	0.812	14.687

Device	Routing	Invert	Outlet Devices
#1	Primary	437.75'	**18.0" Round RCP_Round 18"** L= 100.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 437.75' / 435.75' S= 0.0200 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Device 1	450.00'	**48.0" x 48.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#3	Secondary	450.25'	**25.0' long x 22.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

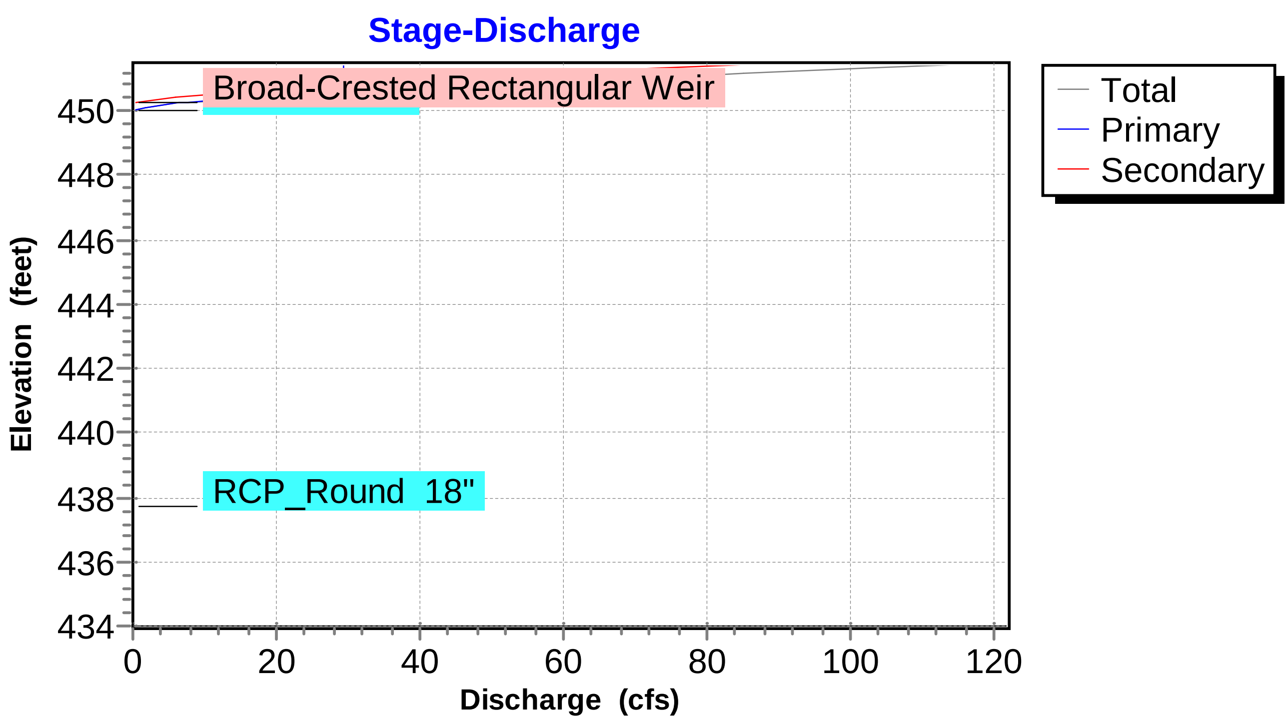
Primary OutFlow Max=6.57 cfs @ 15.46 hrs HW=450.25' (Free Discharge)
↑—**1=RCP_Round 18"** (Passes 6.57 cfs of 28.03 cfs potential flow)
↑—**2=Orifice/Grate** (Weir Controls 6.57 cfs @ 1.64 fps)

Secondary OutFlow Max=0.00 cfs @ 15.46 hrs HW=450.25' (Free Discharge)
↑—**3=Broad-Crested Rectangular Weir** (Weir Controls 0.00 cfs @ 0.07 fps)

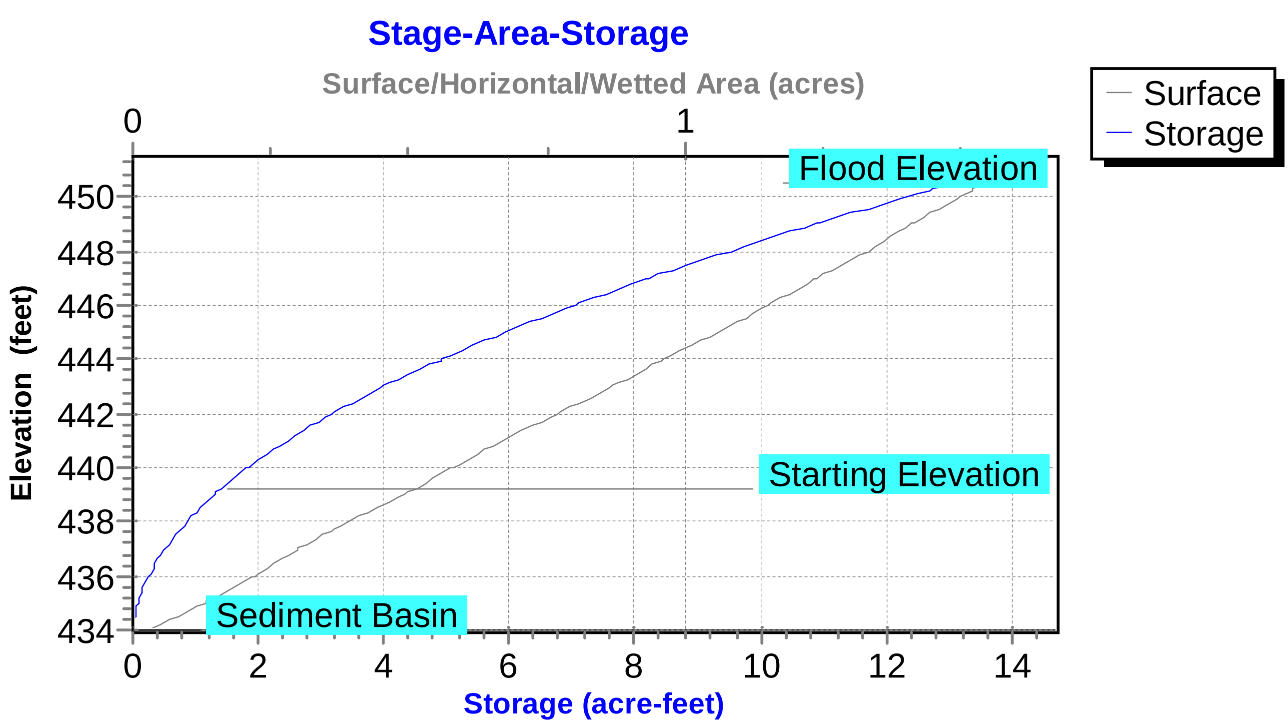
Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)



Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)



Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)



Hydrograph for Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)

Time (hours)	Inflow (cfs)	Storage (acre-feet)	Elevation (feet)	Outflow (cfs)	Primary (cfs)	Secondary (cfs)
0.00	0.00	1.425	439.25	0.00	0.00	0.00
2.00	0.00	1.425	439.25	0.00	0.00	0.00
4.00	0.05	1.427	439.25	0.00	0.00	0.00
6.00	0.31	1.447	439.29	0.00	0.00	0.00
8.00	1.38	1.583	439.55	0.00	0.00	0.00
10.00	4.05	2.006	440.29	0.00	0.00	0.00
12.00	**196.07**	5.787	444.87	0.00	0.00	0.00
14.00	**9.51**	**12.315**	**449.99**	**0.00**	**0.00**	**0.00**
16.00	5.71	**12.699**	**450.24**	**6.26**	**6.26**	**0.00**
18.00	4.34	12.626	450.19	4.68	4.68	0.00
20.00	3.19	12.572	450.16	3.51	3.51	0.00
22.00	2.86	12.545	450.14	2.93	2.93	0.00
24.00	2.63	12.534	450.13	2.69	2.69	0.00
26.00	0.00	12.373	450.03	0.42	0.42	0.00
28.00	0.00	12.341	450.00	0.07	0.07	0.00
30.00	0.00	12.336	450.00	0.01	0.01	0.00
32.00	0.00	12.335	450.00	0.00	0.00	0.00
34.00	0.00	12.335	450.00	0.00	0.00	0.00
36.00	0.00	12.335	450.00	0.00	0.00	0.00
38.00	0.00	12.335	450.00	0.00	0.00	0.00
40.00	0.00	12.335	450.00	0.00	0.00	0.00
42.00	0.00	12.335	450.00	0.00	0.00	0.00
44.00	0.00	12.335	450.00	0.00	0.00	0.00
46.00	0.00	12.335	450.00	0.00	0.00	0.00
48.00	0.00	12.335	450.00	0.00	0.00	0.00
50.00	0.00	12.335	450.00	0.00	0.00	0.00
52.00	0.00	12.335	450.00	0.00	0.00	0.00
54.00	0.00	12.335	450.00	0.00	0.00	0.00
56.00	0.00	12.335	450.00	0.00	0.00	0.00
58.00	0.00	12.335	450.00	0.00	0.00	0.00
60.00	0.00	12.335	450.00	0.00	0.00	0.00
62.00	0.00	12.335	450.00	0.00	0.00	0.00
64.00	0.00	12.335	450.00	0.00	0.00	0.00
66.00	0.00	12.335	450.00	0.00	0.00	0.00
68.00	0.00	12.335	450.00	0.00	0.00	0.00
70.00	0.00	12.335	450.00	0.00	0.00	0.00
72.00	0.00	12.335	450.00	0.00	0.00	0.00

Stage-Discharge for Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)

Elevation (feet)	Discharge (cfs)	Primary (cfs)	Secondary (cfs)
434.00	0.00	0.00	0.00
434.20	0.00	0.00	0.00
434.40	0.00	0.00	0.00
434.60	0.00	0.00	0.00
434.80	0.00	0.00	0.00
435.00	0.00	0.00	0.00
435.20	0.00	0.00	0.00
435.40	0.00	0.00	0.00
435.60	0.00	0.00	0.00
435.80	0.00	0.00	0.00
436.00	0.00	0.00	0.00
436.20	0.00	0.00	0.00
436.40	0.00	0.00	0.00
436.60	0.00	0.00	0.00
436.80	0.00	0.00	0.00
437.00	0.00	0.00	0.00
437.20	0.00	0.00	0.00
437.40	0.00	0.00	0.00
437.60	0.00	0.00	0.00
437.80	0.00	0.00	0.00
438.00	0.00	0.00	0.00
438.20	0.00	0.00	0.00
438.40	0.00	0.00	0.00
438.60	0.00	0.00	0.00
438.80	0.00	0.00	0.00
439.00	0.00	0.00	0.00
439.20	0.00	0.00	0.00
439.40	0.00	0.00	0.00
439.60	0.00	0.00	0.00
439.80	0.00	0.00	0.00
440.00	0.00	0.00	0.00
440.20	0.00	0.00	0.00
440.40	0.00	0.00	0.00
440.60	0.00	0.00	0.00
440.80	0.00	0.00	0.00
441.00	0.00	0.00	0.00
441.20	0.00	0.00	0.00
441.40	0.00	0.00	0.00
441.60	0.00	0.00	0.00
441.80	0.00	0.00	0.00
442.00	0.00	0.00	0.00
442.20	0.00	0.00	0.00
442.40	0.00	0.00	0.00
442.60	0.00	0.00	0.00
442.80	0.00	0.00	0.00
443.00	0.00	0.00	0.00
443.20	0.00	0.00	0.00
443.40	0.00	0.00	0.00
443.60	0.00	0.00	0.00
443.80	0.00	0.00	0.00
444.00	0.00	0.00	0.00
444.20	0.00	0.00	0.00
444.40	0.00	0.00	0.00
444.60	0.00	0.00	0.00
444.80	0.00	0.00	0.00
445.00	0.00	0.00	0.00
445.20	0.00	0.00	0.00
445.40	0.00	0.00	0.00
445.60	0.00	0.00	0.00
445.80	0.00	0.00	0.00
446.00	0.00	0.00	0.00
446.20	0.00	0.00	0.00
446.40	0.00	0.00	0.00
446.60	0.00	0.00	0.00
446.80	0.00	0.00	0.00
447.00	0.00	0.00	0.00
447.20	0.00	0.00	0.00
447.40	0.00	0.00	0.00
447.60	0.00	0.00	0.00
447.80	0.00	0.00	0.00
448.00	0.00	0.00	0.00
448.20	0.00	0.00	0.00
448.40	0.00	0.00	0.00
448.60	0.00	0.00	0.00
448.80	0.00	0.00	0.00
449.00	0.00	0.00	0.00
449.20	0.00	0.00	0.00
449.40	0.00	0.00	0.00
449.60	0.00	0.00	0.00
449.80	0.00	0.00	0.00
450.00	0.00	0.00	0.00
450.20	4.68	4.68	0.00
450.40	17.13	13.24	3.89
450.60	38.27	24.32	13.95
450.80	56.14	28.61	27.53
451.00	71.93	28.82	43.11
451.20	89.97	29.03	60.94
451.40	**110.55**	**29.24**	**81.32**

Stage-Area-Storage for Pond 5P: Sediment Pond (WSE 439.25) Primary Spillway Design (No WQ)

Elevation (feet)	Surface (acres)	Storage (acre-feet)	Elevation (feet)	Surface (acres)	Storage (acre-feet)
434.00	0.030	0.000	444.80	1.040	5.715
434.20	0.050	0.008	445.00	1.060	5.925
434.40	0.070	0.020	445.20	1.078	6.139
434.60	0.090	0.036	445.40	1.096	6.356
434.80	0.110	0.056	445.60	1.114	6.577
435.00	0.130	0.080	445.80	1.132	6.802
435.20	0.148	0.108	446.00	1.150	7.030
435.40	0.166	0.139	446.20	1.168	7.262
435.60	0.184	0.174	446.40	1.186	7.497
435.80	0.202	0.213	446.60	1.204	7.736
436.00	0.220	0.255	446.80	1.222	7.979
436.20	0.236	0.301	447.00	1.240	8.225
436.40	0.252	0.349	447.20	1.258	8.475
436.60	0.268	0.401	447.40	1.276	8.728
436.80	0.284	0.457	447.60	1.294	8.985
437.00	0.300	0.515	447.80	1.312	9.246
437.20	0.318	0.577	448.00	1.330	9.510
437.40	0.336	0.642	448.20	1.346	9.778
437.60	0.354	0.711	448.40	1.362	10.048
437.80	0.372	0.784	448.60	1.378	10.322
438.00	0.390	0.860	448.80	1.394	10.600
438.20	0.410	0.940	449.00	1.410	10.880
438.40	0.430	1.024	449.20	1.428	11.164
438.60	0.450	1.112	449.40	1.446	11.451
438.80	0.470	1.204	449.60	1.464	11.742
439.00	0.490	1.300	449.80	1.482	12.037
439.20	0.508	1.400	450.00	1.500	12.335
439.40	0.526	1.503	450.20	1.516	12.637
439.60	0.544	1.610	450.40	1.532	12.941
439.80	0.562	1.721	450.60	1.548	13.249
440.00	0.580	1.835	450.80	1.564	13.561
440.20	0.598	1.953	451.00	1.580	13.875
440.40	0.616	2.074	451.20	1.616	14.195
440.60	0.634	2.199	451.40	**1.652**	**14.521**
440.80	0.652	2.328			
441.00	0.670	2.460			
441.20	0.690	2.596			
441.40	0.710	2.736			
441.60	0.730	2.880			
441.80	0.750	3.028			
442.00	0.770	3.180			
442.20	0.790	3.336			
442.40	0.810	3.496			
442.60	0.830	3.660			
442.80	0.850	3.828			
443.00	0.870	4.000			
443.20	0.888	4.176			
443.40	0.906	4.355			
443.60	0.924	4.538			
443.80	0.942	4.725			
444.00	0.960	4.915			
444.20	0.980	5.109			
444.40	1.000	5.307			
444.60	1.020	5.509			

Summary for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Inflow Area = 31.050 ac, 4.83% Impervious, Inflow Depth = 5.62" for 100yr-24hr event
Inflow = 208.46 cfs @ 12.05 hrs, Volume= 14.532 af
Outflow = 5.64 cfs @ 16.04 hrs, Volume= 13.573 af, Atten= 97%, Lag= 239.8 min
Primary = 5.64 cfs @ 16.04 hrs, Volume= 13.573 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Starting Elev= 439.25' Surf.Area= 0.512 ac Storage= 1.425 af
Peak Elev= 449.44' @ 16.04 hrs Surf.Area= 1.450 ac Storage= 11.515 af (10.090 af above start)
Flood Elev= 450.50' Surf.Area= 1.540 ac Storage= 13.095 af (11.670 af above start)

Plug-Flow detention time= 1,243.7 min calculated for 12.146 af (84% of inflow)
Center-of-Mass det. time= 1,044.2 min (1,852.4 - 808.2)

Volume	Invert	Avail.Storage	Storage Description
#1	434.00'	14.687 af	**Sediment Basin (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
434.00	0.030	0.000	0.000
435.00	0.130	0.080	0.080
436.00	0.220	0.175	0.255
437.00	0.300	0.260	0.515
438.00	0.390	0.345	0.860
439.00	0.490	0.440	1.300
440.00	0.580	0.535	1.835
441.00	0.670	0.625	2.460
442.00	0.770	0.720	3.180
443.00	0.870	0.820	4.000
444.00	0.960	0.915	4.915
445.00	1.060	1.010	5.925
446.00	1.150	1.105	7.030
447.00	1.240	1.195	8.225
448.00	1.330	1.285	9.510
449.00	1.410	1.370	10.880
450.00	1.500	1.455	12.335
451.00	1.580	1.540	13.875
451.50	1.670	0.812	14.687

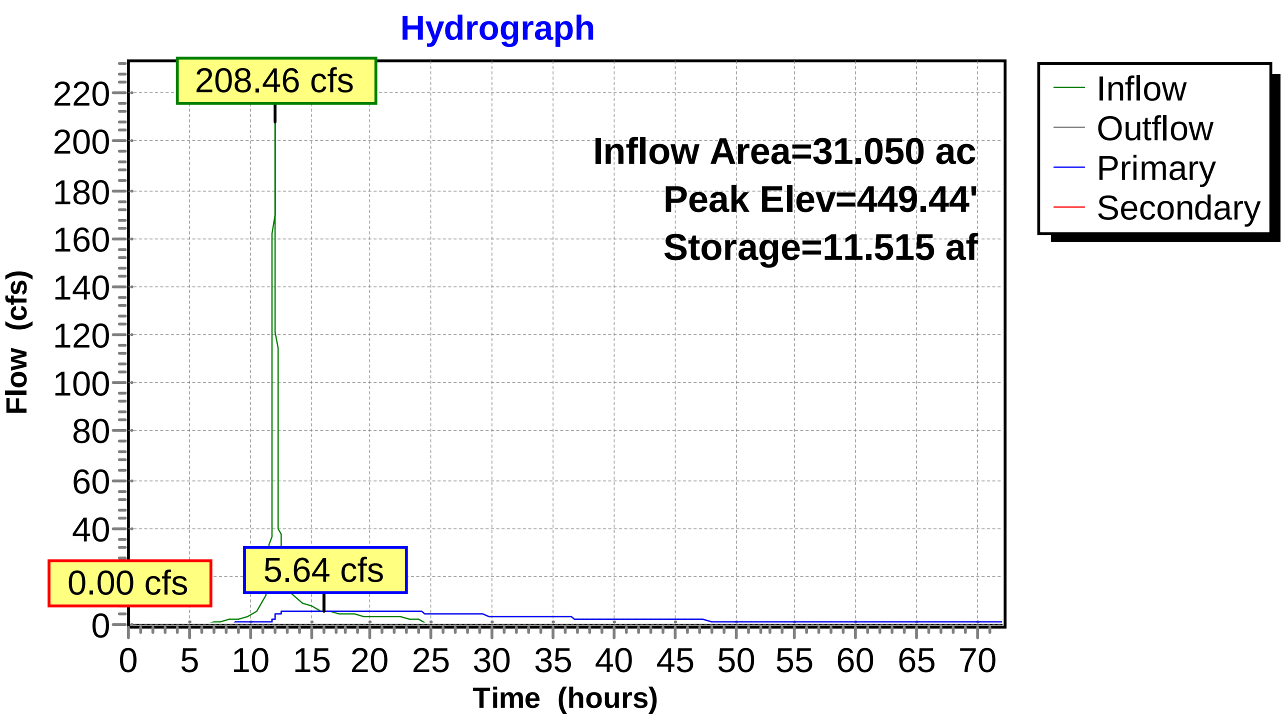
Device	Routing	Invert	Outlet Devices
#1	Primary	438.75'	**18.0" Round RCP_Round 18"** L= 100.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 438.75' / 436.75' S= 0.0200 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Device 1	439.25'	**0.734 cfs Constant Flow/Skimmer** Phase-In= 0.50'
#3	Device 1	441.25'	**1.5" Vert. Orifice/Grate X 5.00 columns** X 8 rows with 12.0" cc spacing C= 0.600
#4	Device 1	450.00'	**48.0" x 48.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

#5	Secondary	450.25'	**25.0' long x 22.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

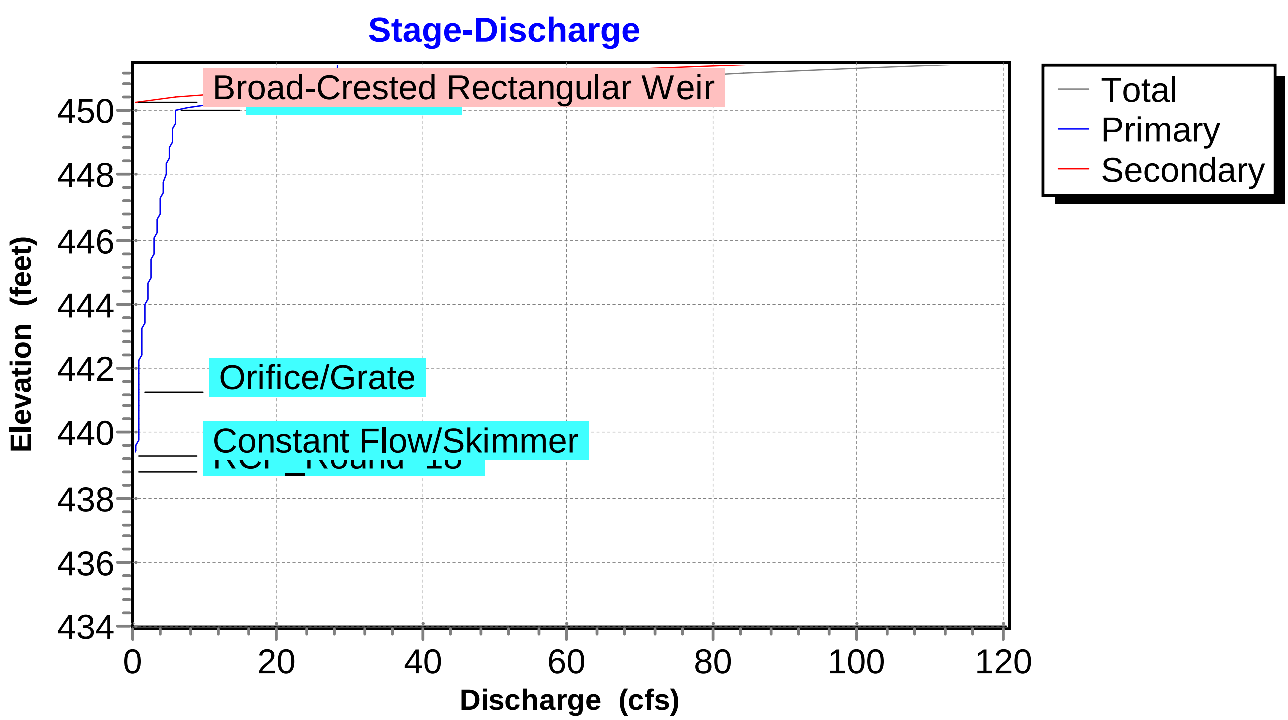
Primary OutFlow Max=5.64 cfs @ 16.04 hrs HW=449.44' (Free Discharge)
1=RCP_Round 18" (Passes 5.64 cfs of 26.01 cfs potential flow)
2=Constant Flow/Skimmer (Constant Controls 0.73 cfs)
3=Orifice/Grate (Orifice Controls 4.90 cfs @ 9.98 fps)
4=Orifice/Grate (Controls 0.00 cfs)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=439.25' (Free Discharge)
5=Broad-Crested Rectangular Weir (Controls 0.00 cfs)

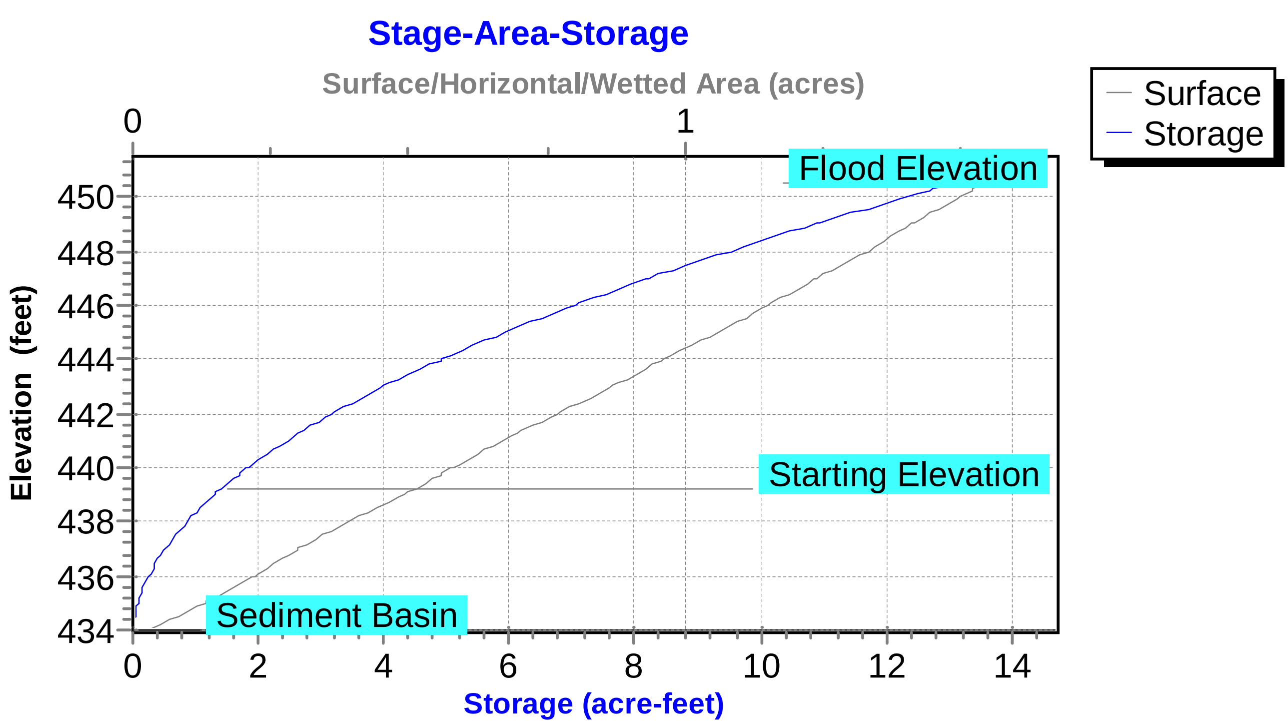
Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Hydrograph for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Time (hours)	Inflow (cfs)	Storage (acre-feet)	Elevation (feet)	Outflow (cfs)	Primary (cfs)	Secondary (cfs)
0.00	0.00	1.425	439.25	0.00	0.00	**0.00**
2.00	0.00	1.425	439.25	0.00	0.00	0.00
4.00	0.05	1.427	439.25	0.01	0.01	0.00
6.00	0.31	1.443	439.28	0.05	0.05	0.00
8.00	1.38	1.551	439.49	0.35	0.35	0.00
10.00	4.05	1.873	440.07	0.73	0.73	0.00
12.00	**196.07**	5.514	444.60	2.21	2.21	0.00
14.00	**9.51**	11.227	449.24	5.51	5.51	0.00
16.00	5.71	**11.515**	**449.44**	**5.63**	**5.63**	0.00
18.00	4.34	**11.401**	**449.37**	**5.59**	**5.59**	0.00
20.00	3.19	11.110	449.16	5.45	5.45	0.00
22.00	2.86	10.717	448.88	5.26	5.26	0.00
24.00	2.63	10.319	448.60	5.04	5.04	0.00
26.00	0.00	9.575	448.05	4.55	4.55	0.00
28.00	0.00	8.854	447.50	4.16	4.16	0.00
30.00	0.00	8.205	446.98	3.75	3.75	0.00
32.00	0.00	7.612	446.50	3.42	3.42	0.00
34.00	0.00	7.079	446.04	3.08	3.08	0.00
36.00	0.00	6.590	445.61	2.83	2.83	0.00
38.00	0.00	6.149	445.21	2.51	2.51	0.00
40.00	0.00	5.748	444.83	2.33	2.33	0.00
42.00	0.00	5.379	444.47	2.12	2.12	0.00
44.00	0.00	5.049	444.14	1.90	1.90	0.00
46.00	0.00	4.745	443.82	1.78	1.78	0.00
48.00	0.00	4.463	443.52	1.63	1.63	0.00
50.00	0.00	4.211	443.24	1.43	1.43	0.00
52.00	0.00	3.981	442.98	1.36	1.36	0.00
54.00	0.00	3.763	442.72	1.27	1.27	0.00
56.00	0.00	3.561	442.48	1.17	1.17	0.00
58.00	0.00	3.378	442.25	1.03	1.03	0.00
60.00	0.00	3.212	442.04	0.99	0.99	0.00
62.00	0.00	3.052	441.83	0.95	0.95	0.00
64.00	0.00	2.900	441.63	0.90	0.90	0.00
66.00	0.00	2.757	441.43	0.83	0.83	0.00
68.00	0.00	2.628	441.25	0.73	0.73	0.00
70.00	0.00	2.506	441.07	0.73	0.73	0.00
72.00	0.00	2.385	440.89	0.73	0.73	0.00

Stage-Discharge for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Elevation (feet)	Discharge (cfs)	Primary (cfs)	Secondary (cfs)
434.00	0.00	0.00	0.00
434.20	0.00	0.00	0.00
434.40	0.00	0.00	0.00
434.60	0.00	0.00	0.00
434.80	0.00	0.00	0.00
435.00	0.00	0.00	0.00
435.20	0.00	0.00	0.00
435.40	0.00	0.00	0.00
435.60	0.00	0.00	0.00
435.80	0.00	0.00	0.00
436.00	0.00	0.00	0.00
436.20	0.00	0.00	0.00
436.40	0.00	0.00	0.00
436.60	0.00	0.00	0.00
436.80	0.00	0.00	0.00
437.00	0.00	0.00	0.00
437.20	0.00	0.00	0.00
437.40	0.00	0.00	0.00
437.60	0.00	0.00	0.00
437.80	0.00	0.00	0.00
438.00	0.00	0.00	0.00
438.20	0.00	0.00	0.00
438.40	0.00	0.00	0.00
438.60	0.00	0.00	0.00
438.80	0.00	0.00	0.00
439.00	0.00	0.00	0.00
439.20	0.00	0.00	0.00
439.40	0.22	0.22	0.00
439.60	0.51	0.51	0.00
439.80	0.73	0.73	0.00
440.00	0.73	0.73	0.00
440.20	0.73	0.73	0.00
440.40	0.73	0.73	0.00
440.60	0.73	0.73	0.00
440.80	0.73	0.73	0.00
441.00	0.73	0.73	0.00
441.20	0.73	0.73	0.00
441.40	0.82	0.82	0.00
441.60	0.89	0.89	0.00
441.80	0.94	0.94	0.00
442.00	0.98	0.98	0.00
442.20	1.01	1.01	0.00
442.40	1.13	1.13	0.00
442.60	1.23	1.23	0.00
442.80	1.30	1.30	0.00
443.00	1.36	1.36	0.00
443.20	1.42	1.42	0.00
443.40	1.56	1.56	0.00
443.60	1.67	1.67	0.00
443.80	1.77	1.77	0.00
444.00	1.85	1.85	0.00
444.20	1.92	1.92	0.00
444.40	2.08	2.08	0.00
444.60	2.21	2.21	0.00
444.80	2.32	2.32	0.00
445.00	2.41	2.41	0.00
445.20	2.50	2.50	0.00
445.40	2.67	2.67	0.00
445.60	2.82	2.82	0.00
445.80	2.94	2.94	0.00
446.00	3.05	3.05	0.00
446.20	3.16	3.16	0.00
446.40	3.34	3.34	0.00
446.60	3.50	3.50	0.00
446.80	3.64	3.64	0.00
447.00	3.76	3.76	0.00
447.20	3.87	3.87	0.00
447.40	4.07	4.07	0.00
447.60	4.24	4.24	0.00
447.80	4.39	4.39	0.00
448.00	4.52	4.52	0.00
448.20	4.65	4.65	0.00
448.40	4.85	4.85	0.00
448.60	5.04	5.04	0.00
448.80	5.20	5.20	0.00
449.00	5.34	5.34	0.00
449.20	5.48	5.48	0.00
449.40	5.61	5.61	0.00
449.60	5.73	5.73	0.00
449.80	5.85	5.85	0.00
450.00	5.97	5.97	0.00
450.20	10.76	10.76	0.00
450.40	23.32	19.43	3.89
450.60	41.27	27.32	13.95
450.80	55.07	27.54	27.53
451.00	70.87	27.75	43.11
451.20	88.91	27.97	60.94
451.40	**109.50**	**28.19**	**81.32**

Stage-Area-Storage for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Elevation (feet)	Surface (acres)	Storage (acre-feet)	Elevation (feet)	Surface (acres)	Storage (acre-feet)
434.00	0.030	0.000	444.80	1.040	5.715
434.20	0.050	0.008	445.00	1.060	5.925
434.40	0.070	0.020	445.20	1.078	6.139
434.60	0.090	0.036	445.40	1.096	6.356
434.80	0.110	0.056	445.60	1.114	6.577
435.00	0.130	0.080	445.80	1.132	6.802
435.20	0.148	0.108	446.00	1.150	7.030
435.40	0.166	0.139	446.20	1.168	7.262
435.60	0.184	0.174	446.40	1.186	7.497
435.80	0.202	0.213	446.60	1.204	7.736
436.00	0.220	0.255	446.80	1.222	7.979
436.20	0.236	0.301	447.00	1.240	8.225
436.40	0.252	0.349	447.20	1.258	8.475
436.60	0.268	0.401	447.40	1.276	8.728
436.80	0.284	0.457	447.60	1.294	8.985
437.00	0.300	0.515	447.80	1.312	9.246
437.20	0.318	0.577	448.00	1.330	9.510
437.40	0.336	0.642	448.20	1.346	9.778
437.60	0.354	0.711	448.40	1.362	10.048
437.80	0.372	0.784	448.60	1.378	10.322
438.00	0.390	0.860	448.80	1.394	10.600
438.20	0.410	0.940	449.00	1.410	10.880
438.40	0.430	1.024	449.20	1.428	11.164
438.60	0.450	1.112	449.40	1.446	11.451
438.80	0.470	1.204	449.60	1.464	11.742
439.00	0.490	1.300	449.80	1.482	12.037
439.20	0.508	1.400	450.00	1.500	12.335
439.40	0.526	1.503	450.20	1.516	12.637
439.60	0.544	1.610	450.40	1.532	12.941
439.80	0.562	1.721	450.60	1.548	13.249
440.00	0.580	1.835	450.80	1.564	13.561
440.20	0.598	1.953	451.00	1.580	13.875
440.40	0.616	2.074	451.20	1.616	14.195
440.60	0.634	2.199	451.40	**1.652**	**14.521**
440.80	0.652	2.328			
441.00	0.670	2.460			
441.20	0.690	2.596			
441.40	0.710	2.736			
441.60	0.730	2.880			
441.80	0.750	3.028			
442.00	0.770	3.180			
442.20	0.790	3.336			
442.40	0.810	3.496			
442.60	0.830	3.660			
442.80	0.850	3.828			
443.00	0.870	4.000			
443.20	0.888	4.176			
443.40	0.906	4.355			
443.60	0.924	4.538			
443.80	0.942	4.725			
444.00	0.960	4.915			
444.20	0.980	5.109			
444.40	1.000	5.307			
444.60	1.020	5.509			

Summary for Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)

Inflow Area = 31.050 ac, 4.83% Impervious, Inflow Depth = 5.62" for 100yr-24hr event
Inflow = 208.46 cfs @ 12.05 hrs, Volume= 14.532 af
Outflow = 5.88 cfs @ 15.89 hrs, Volume= 3.245 af, Atten= 97%, Lag= 230.5 min
Primary = 5.88 cfs @ 15.89 hrs, Volume= 3.245 af

Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Starting Elev= 439.25' Surf.Area= 0.512 ac Storage= 1.425 af
Peak Elev= 450.45' @ 15.89 hrs Surf.Area= 1.536 ac Storage= 13.013 af (11.588 af above start)
Flood Elev= 450.50' Surf.Area= 1.540 ac Storage= 13.095 af (11.670 af above start)

Plug-Flow detention time= 658.8 min calculated for 1.820 af (13% of inflow)
Center-of-Mass det. time= 339.6 min (1,147.9 - 808.2)

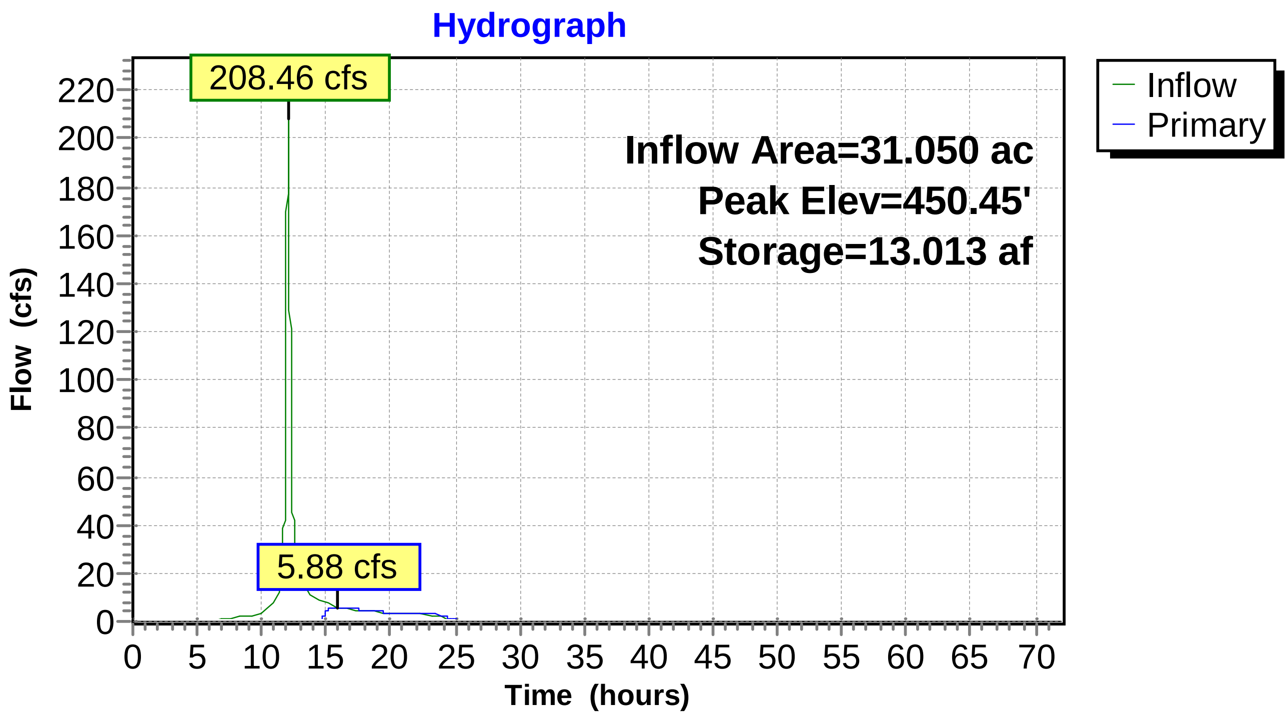
Volume	Invert	Avail.Storage	Storage Description
#1	434.00'	14.687 af	**Sediment Basin (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
434.00	0.030	0.000	0.000
435.00	0.130	0.080	0.080
436.00	0.220	0.175	0.255
437.00	0.300	0.260	0.515
438.00	0.390	0.345	0.860
439.00	0.490	0.440	1.300
440.00	0.580	0.535	1.835
441.00	0.670	0.625	2.460
442.00	0.770	0.720	3.180
443.00	0.870	0.820	4.000
444.00	0.960	0.915	4.915
445.00	1.060	1.010	5.925
446.00	1.150	1.105	7.030
447.00	1.240	1.195	8.225
448.00	1.330	1.285	9.510
449.00	1.410	1.370	10.880
450.00	1.500	1.455	12.335
451.00	1.580	1.540	13.875
451.50	1.670	0.812	14.687

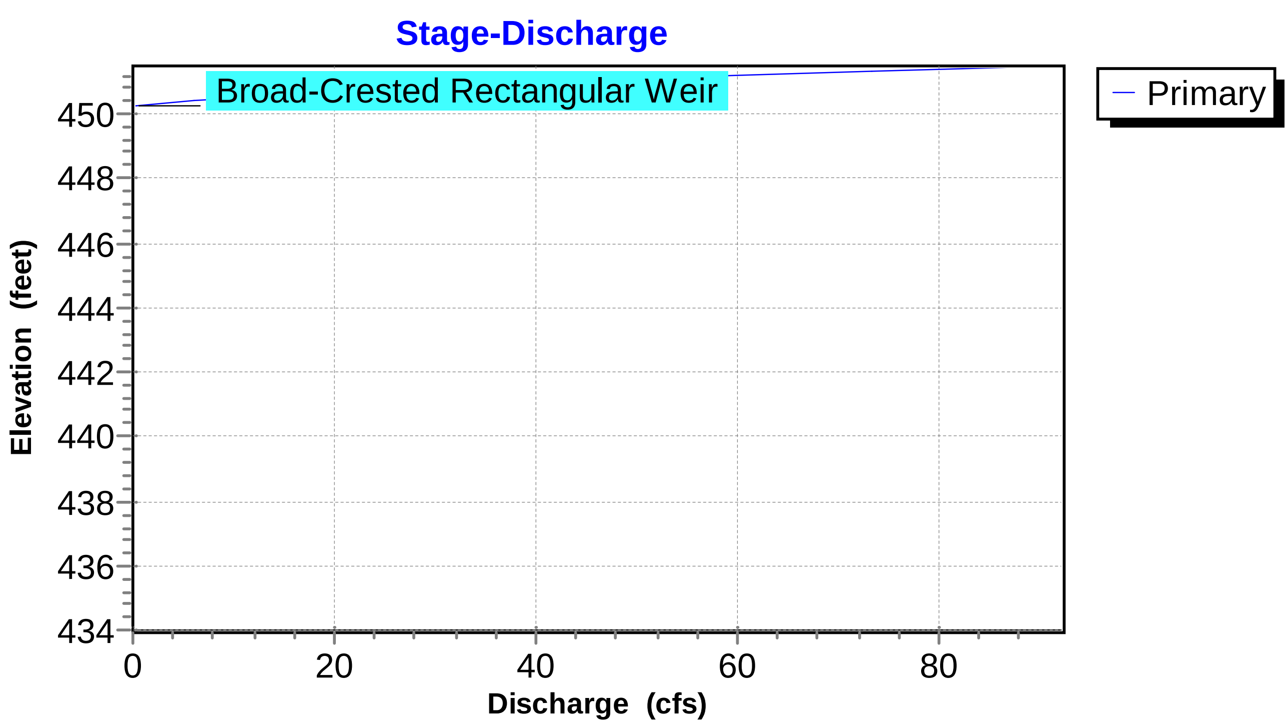
Device	Routing	Invert	Outlet Devices
#1	Primary	450.25'	**25.0' long x 22.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.68 2.70 2.70 2.64 2.63 2.64 2.64 2.63

Primary OutFlow Max=5.84 cfs @ 15.89 hrs HW=450.45' (Free Discharge)
└**1=Broad-Crested Rectangular Weir** (Weir Controls 5.84 cfs @ 1.19 fps)

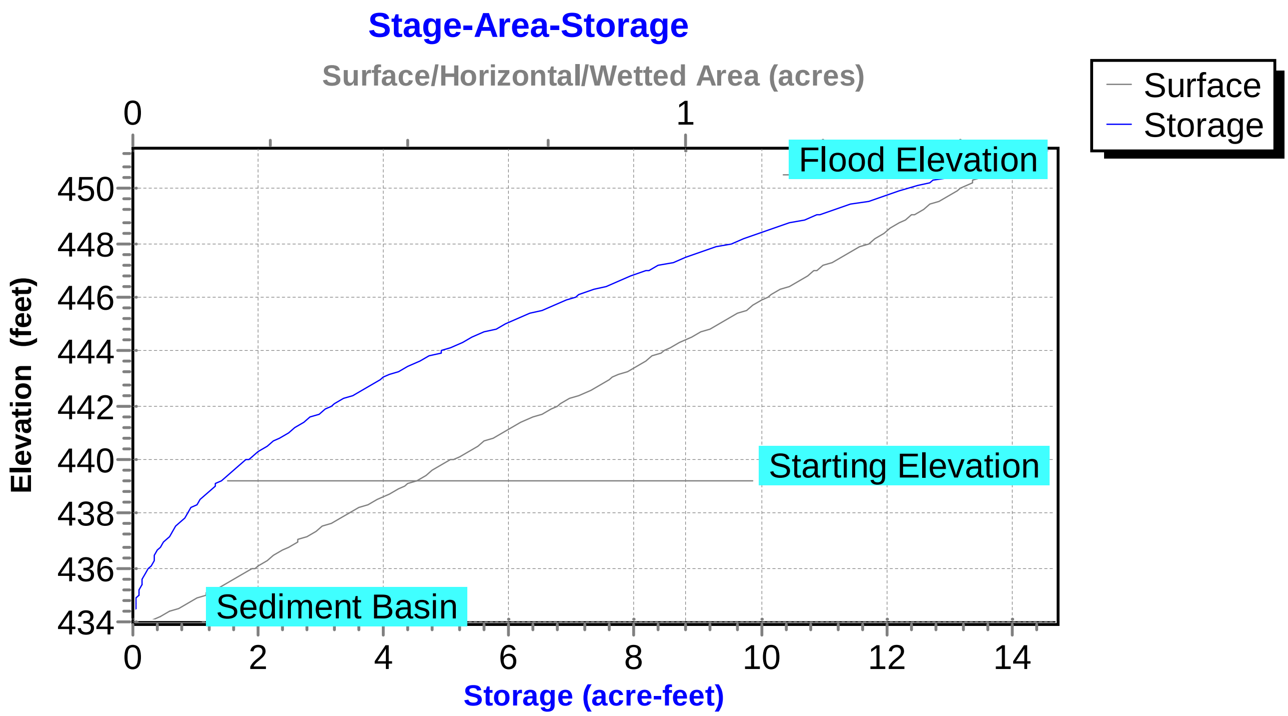
Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)



Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)



Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)



Hydrograph for Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)

Time (hours)	Inflow (cfs)	Storage (acre-feet)	Elevation (feet)	Primary (cfs)
0.00	0.00	1.425	439.25	0.00
2.00	0.00	1.425	439.25	0.00
4.00	0.05	1.427	439.25	0.00
6.00	0.31	1.447	439.29	0.00
8.00	1.38	1.583	439.55	0.00
10.00	4.05	2.006	440.29	0.00
12.00	**196.07**	5.787	444.87	0.00
14.00	**9.51**	**12.315**	**449.99**	**0.00**
16.00	5.71	**13.012**	**450.45**	**5.87**
18.00	4.34	12.956	450.41	4.67
20.00	3.19	12.902	450.37	3.51
22.00	2.86	12.875	450.36	2.93
24.00	2.63	12.864	450.35	2.69
26.00	0.00	12.742	450.27	0.20
28.00	0.00	12.722	450.26	0.06
30.00	0.00	12.715	450.25	0.02
32.00	0.00	12.713	450.25	0.01
34.00	0.00	12.713	450.25	0.00
36.00	0.00	12.713	450.25	0.00
38.00	0.00	12.713	450.25	0.00
40.00	0.00	12.713	450.25	0.00
42.00	0.00	12.713	450.25	0.00
44.00	0.00	12.713	450.25	0.00
46.00	0.00	12.713	450.25	0.00
48.00	0.00	12.713	450.25	0.00
50.00	0.00	12.713	450.25	0.00
52.00	0.00	12.713	450.25	0.00
54.00	0.00	12.713	450.25	0.00
56.00	0.00	12.713	450.25	0.00
58.00	0.00	12.713	450.25	0.00
60.00	0.00	12.713	450.25	0.00
62.00	0.00	12.713	450.25	0.00
64.00	0.00	12.713	450.25	0.00
66.00	0.00	12.713	450.25	0.00
68.00	0.00	12.713	450.25	0.00
70.00	0.00	12.713	450.25	0.00
72.00	0.00	12.713	450.25	0.00

Stage-Discharge for Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)

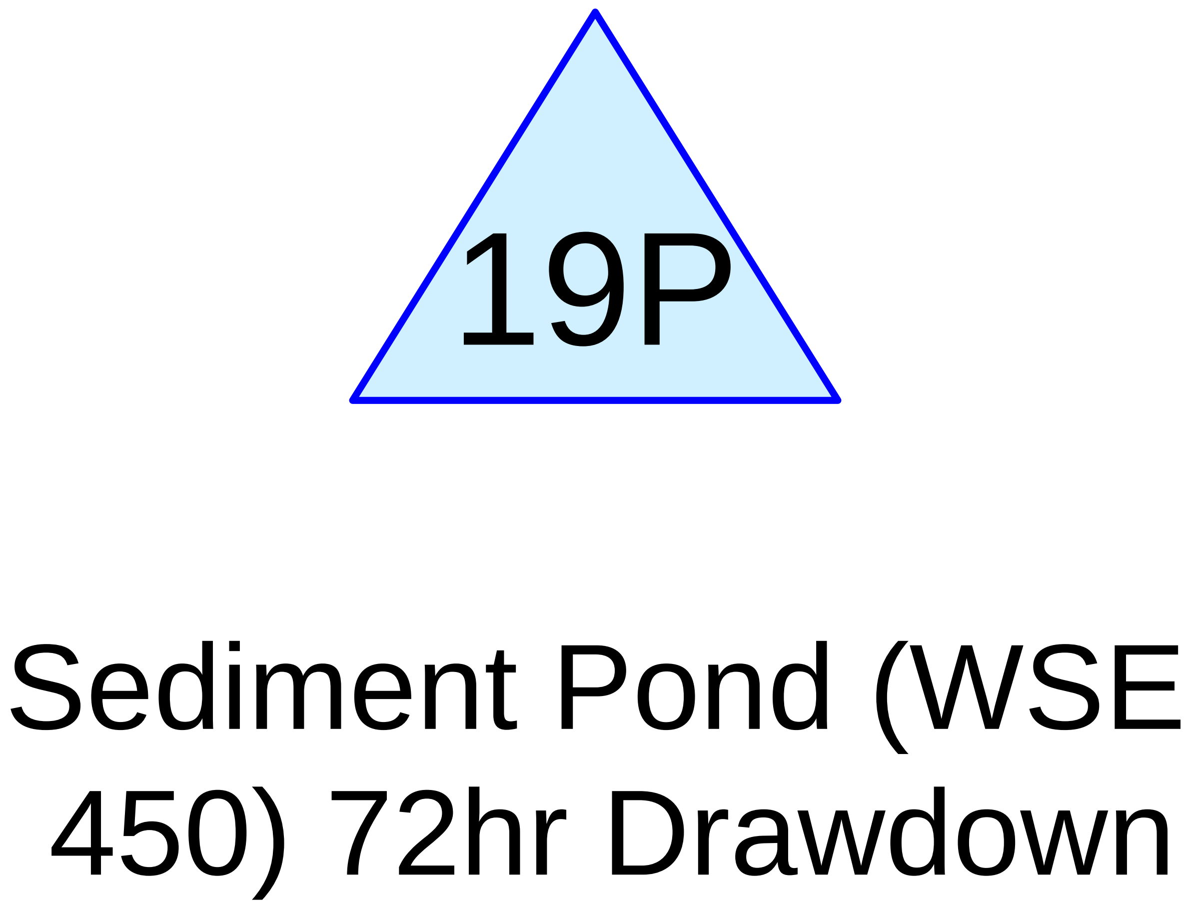
Elevation (feet)	Primary (cfs)	Elevation (feet)	Primary (cfs)
434.00	0.00	444.80	0.00
434.20	0.00	445.00	0.00
434.40	0.00	445.20	0.00
434.60	0.00	445.40	0.00
434.80	0.00	445.60	0.00
435.00	0.00	445.80	0.00
435.20	0.00	446.00	0.00
435.40	0.00	446.20	0.00
435.60	0.00	446.40	0.00
435.80	0.00	446.60	0.00
436.00	0.00	446.80	0.00
436.20	0.00	447.00	0.00
436.40	0.00	447.20	0.00
436.60	0.00	447.40	0.00
436.80	0.00	447.60	0.00
437.00	0.00	447.80	0.00
437.20	0.00	448.00	0.00
437.40	0.00	448.20	0.00
437.60	0.00	448.40	0.00
437.80	0.00	448.60	0.00
438.00	0.00	448.80	0.00
438.20	0.00	449.00	0.00
438.40	0.00	449.20	0.00
438.60	0.00	449.40	0.00
438.80	0.00	449.60	0.00
439.00	0.00	449.80	0.00
439.20	0.00	450.00	0.00
439.40	0.00	450.20	0.00
439.60	0.00	450.40	3.89
439.80	0.00	450.60	13.95
440.00	0.00	450.80	27.53
440.20	0.00	451.00	43.11
440.40	0.00	451.20	60.94
440.60	0.00	451.40	**81.32**
440.80	0.00		
441.00	0.00		
441.20	0.00		
441.40	0.00		
441.60	0.00		
441.80	0.00		
442.00	0.00		
442.20	0.00		
442.40	0.00		
442.60	0.00		
442.80	0.00		
443.00	0.00		
443.20	0.00		
443.40	0.00		
443.60	0.00		
443.80	0.00		
444.00	0.00		
444.20	0.00		
444.40	0.00		
444.60	0.00		

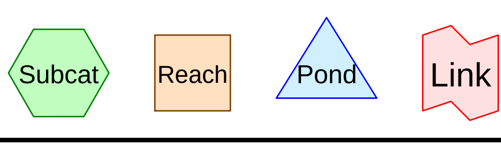
Stage-Area-Storage for Pond 27P: Sediment Pond (WSE 439.25) Emergency Design (No WQ No Primary)

Elevation (feet)	Surface (acres)	Storage (acre-feet)	Elevation (feet)	Surface (acres)	Storage (acre-feet)
434.00	0.030	0.000	444.80	1.040	5.715
434.20	0.050	0.008	445.00	1.060	5.925
434.40	0.070	0.020	445.20	1.078	6.139
434.60	0.090	0.036	445.40	1.096	6.356
434.80	0.110	0.056	445.60	1.114	6.577
435.00	0.130	0.080	445.80	1.132	6.802
435.20	0.148	0.108	446.00	1.150	7.030
435.40	0.166	0.139	446.20	1.168	7.262
435.60	0.184	0.174	446.40	1.186	7.497
435.80	0.202	0.213	446.60	1.204	7.736
436.00	0.220	0.255	446.80	1.222	7.979
436.20	0.236	0.301	447.00	1.240	8.225
436.40	0.252	0.349	447.20	1.258	8.475
436.60	0.268	0.401	447.40	1.276	8.728
436.80	0.284	0.457	447.60	1.294	8.985
437.00	0.300	0.515	447.80	1.312	9.246
437.20	0.318	0.577	448.00	1.330	9.510
437.40	0.336	0.642	448.20	1.346	9.778
437.60	0.354	0.711	448.40	1.362	10.048
437.80	0.372	0.784	448.60	1.378	10.322
438.00	0.390	0.860	448.80	1.394	10.600
438.20	0.410	0.940	449.00	1.410	10.880
438.40	0.430	1.024	449.20	1.428	11.164
438.60	0.450	1.112	449.40	1.446	11.451
438.80	0.470	1.204	449.60	1.464	11.742
439.00	0.490	1.300	449.80	1.482	12.037
439.20	0.508	1.400	450.00	1.500	12.335
439.40	0.526	1.503	450.20	1.516	12.637
439.60	0.544	1.610	450.40	1.532	12.941
439.80	0.562	1.721	450.60	1.548	13.249
440.00	0.580	1.835	450.80	1.564	13.561
440.20	0.598	1.953	451.00	1.580	13.875
440.40	0.616	2.074	451.20	1.616	14.195
440.60	0.634	2.199	451.40	**1.652**	**14.521**
440.80	0.652	2.328			
441.00	0.670	2.460			
441.20	0.690	2.596			
441.40	0.710	2.736			
441.60	0.730	2.880			
441.80	0.750	3.028			
442.00	0.770	3.180			
442.20	0.790	3.336			
442.40	0.810	3.496			
442.60	0.830	3.660			
442.80	0.850	3.828			
443.00	0.870	4.000			
443.20	0.888	4.176			
443.40	0.906	4.355			
443.60	0.924	4.538			
443.80	0.942	4.725			
444.00	0.960	4.915			
444.20	0.980	5.109			
444.40	1.000	5.307			
444.60	1.020	5.509			

ATTACHMENT D

HydroCAD report for Sediment Basin - 72-hour dry pool dewatering





Routing Diagram for Sediment Basin
Prepared by URS Corporation, Printed 12/23/2016

Summary for Pond 19P: Sediment Pond (WSE 450) 72hr Drawdown

Inflow Area = 31.050 ac, 4.83% Impervious, Inflow Depth = 0.00" for No Flow event
Inflow = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af
Outflow = 5.97 cfs @ 0.00 hrs, Volume= 10.873 af, Atten= 0%, Lag= 0.0 min
Primary = 5.97 cfs @ 0.00 hrs, Volume= 10.873 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Starting Elev= 450.00' Surf.Area= 1.500 ac Storage= 12.335 af
Peak Elev= 450.00' @ 0.00 hrs Surf.Area= 1.500 ac Storage= 12.335 af
Flood Elev= 450.50' Surf.Area= 1.540 ac Storage= 13.095 af (0.760 af above start)

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no inflow)

Volume	Invert	Avail.Storage	Storage Description
#1	434.00'	14.687 af	**Sediment Basin (Prismatic)**Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
434.00	0.030	0.000	0.000
435.00	0.130	0.080	0.080
436.00	0.220	0.175	0.255
437.00	0.300	0.260	0.515
438.00	0.390	0.345	0.860
439.00	0.490	0.440	1.300
440.00	0.580	0.535	1.835
441.00	0.670	0.625	2.460
442.00	0.770	0.720	3.180
443.00	0.870	0.820	4.000
444.00	0.960	0.915	4.915
445.00	1.060	1.010	5.925
446.00	1.150	1.105	7.030
447.00	1.240	1.195	8.225
448.00	1.330	1.285	9.510
449.00	1.410	1.370	10.880
450.00	1.500	1.455	12.335
451.00	1.580	1.540	13.875
451.50	1.670	0.812	14.687

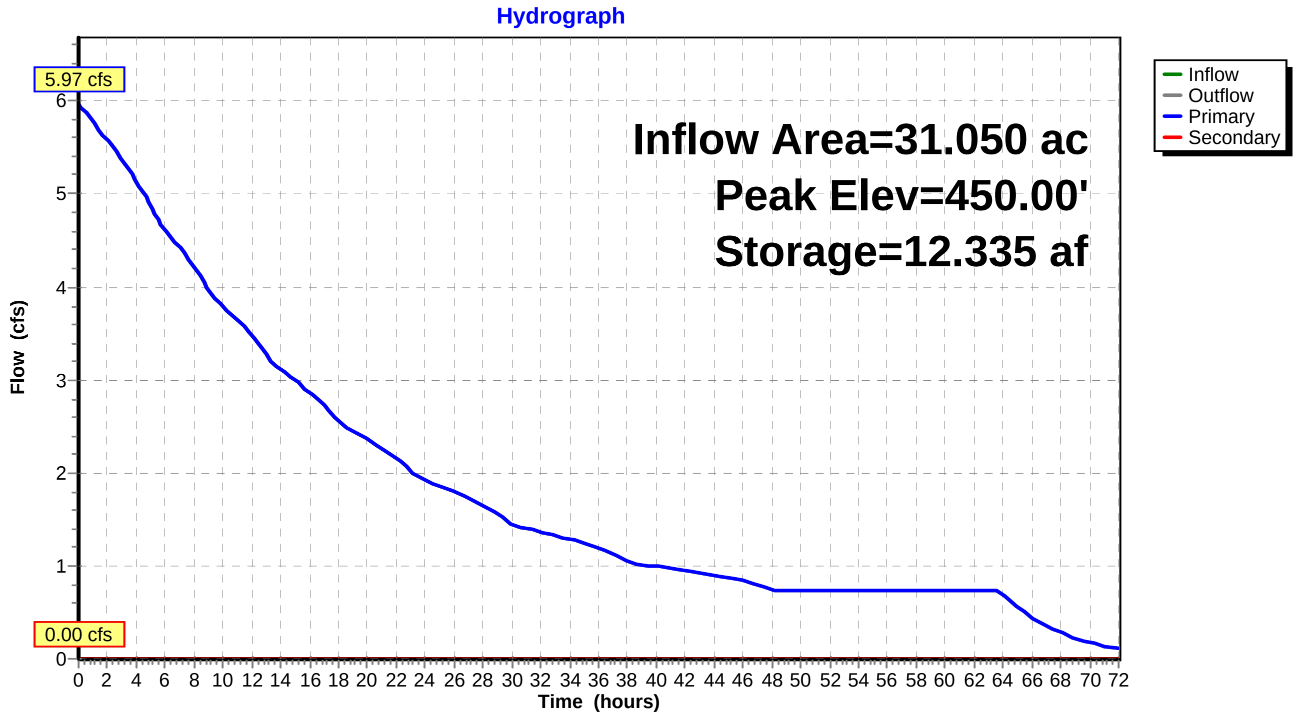
Device	Routing	Invert	Outlet Devices
#1	Primary	438.75'	**18.0" Round RCP_Round 18"** L= 100.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 438.75' / 436.75' S= 0.0200 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Device 1	439.25'	**0.734 cfs Constant Flow/Skimmer** Phase-In= 0.50'
#3	Device 1	441.25'	**1.5" Vert. Orifice/Grate X 5.00 columns** X 8 rows with 12.0" cc spacing C= 0.600
#4	Device 1	450.00'	**48.0" x 48.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#5	Secondary	450.25'	**25.0' long x 10.0' breadth Broad-Crested Rectangular Weir**

Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60
Coef. (English) 2.49 2.56 2.70 2.69 2.68 2.69 2.67 2.64

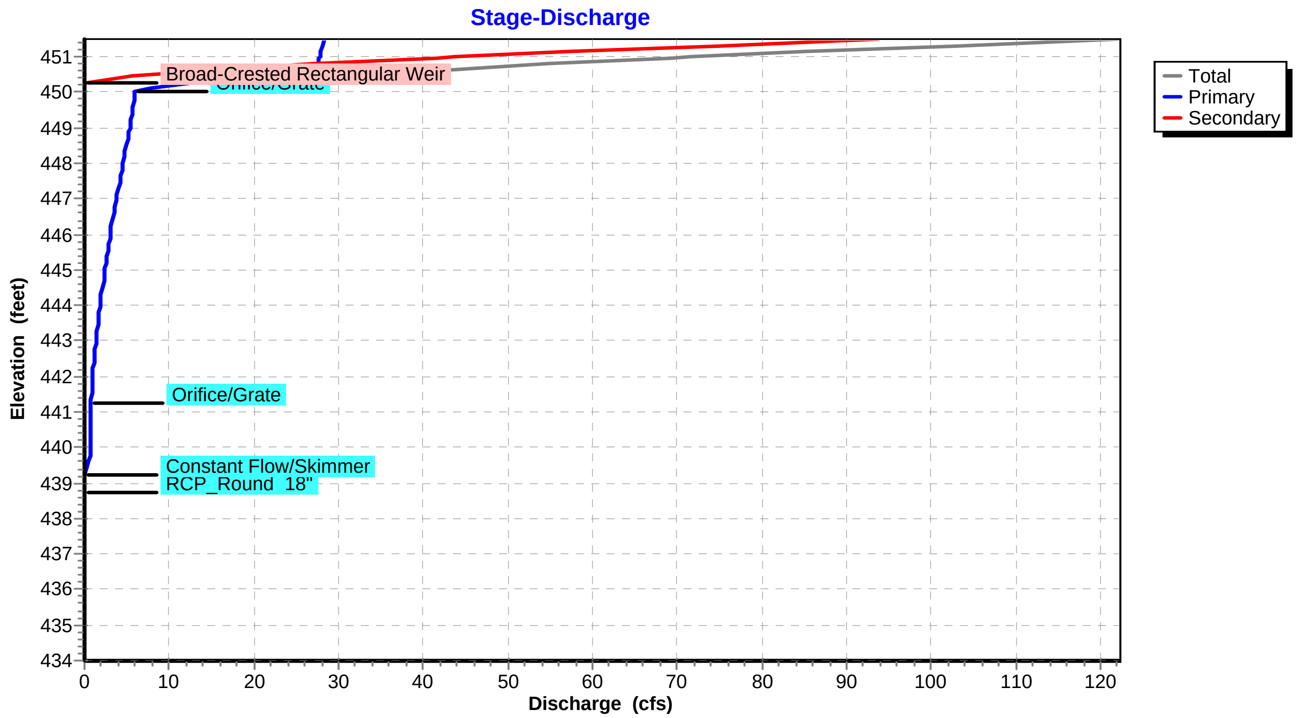
Primary OutFlow Max=5.97 cfs @ 0.00 hrs HW=450.00' (Free Discharge)
1=RCP_Round 18" (Passes 5.97 cfs of 26.64 cfs potential flow)
2=Constant Flow/Skimmer (Constant Controls 0.73 cfs)
3=Orifice/Grate (Orifice Controls 5.23 cfs @ 10.66 fps)
4=Orifice/Grate (Controls 0.00 cfs)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=450.00' (Free Discharge)
5=Broad-Crested Rectangular Weir (Controls 0.00 cfs)

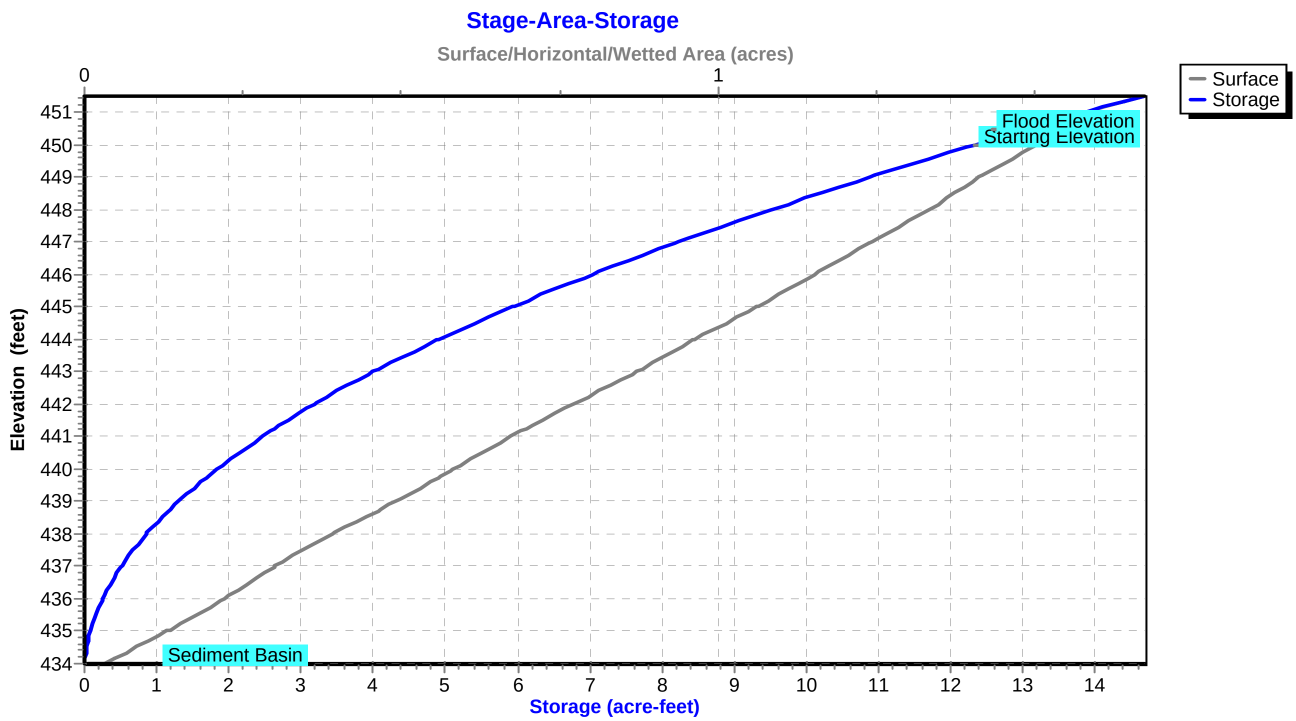
Pond 19P: Sediment Pond (WSE 450) 72hr Drawdown



Pond 19P: Sediment Pond (WSE 450) 72hr Drawdown



Pond 19P: Sediment Pond (WSE 450) 72hr Drawdown



Hydrograph for Pond 19P: Sediment Pond (WSE 450) 72hr Drawdown

Time (hours)	Inflow (cfs)	Storage (acre-feet)	Elevation (feet)	Outflow (cfs)	Primary (cfs)	Secondary (cfs)
0.00	**0.00**	**12.335**	**450.00**	**5.97**	**5.97**	**0.00**
2.00	0.00	11.381	449.35	5.58	5.58	0.00
4.00	0.00	10.494	448.72	5.14	5.14	0.00
6.00	0.00	9.689	448.13	4.61	4.61	0.00
8.00	0.00	8.958	447.58	4.22	4.22	0.00
10.00	0.00	8.299	447.06	3.79	3.79	0.00
12.00	0.00	7.698	446.57	3.48	3.48	0.00
14.00	0.00	7.156	446.11	3.11	3.11	0.00
16.00	0.00	6.662	445.68	2.87	2.87	0.00
18.00	0.00	6.212	445.27	2.56	2.56	0.00
20.00	0.00	5.807	444.89	2.36	2.36	0.00
22.00	0.00	5.433	444.52	2.16	2.16	0.00
24.00	0.00	5.097	444.19	1.92	1.92	0.00
26.00	0.00	4.790	443.87	1.79	1.79	0.00
28.00	0.00	4.504	443.56	1.65	1.65	0.00
30.00	0.00	4.247	443.28	1.45	1.45	0.00
32.00	0.00	4.015	443.02	1.37	1.37	0.00
34.00	0.00	3.795	442.76	1.29	1.29	0.00
36.00	0.00	3.590	442.52	1.19	1.19	0.00
38.00	0.00	3.404	442.29	1.06	1.06	0.00
40.00	0.00	3.237	442.07	0.99	0.99	0.00
42.00	0.00	3.076	441.86	0.95	0.95	0.00
44.00	0.00	2.922	441.66	0.91	0.91	0.00
46.00	0.00	2.777	441.46	0.84	0.84	0.00
48.00	0.00	2.646	441.27	0.75	0.75	0.00
50.00	0.00	2.525	441.10	0.73	0.73	0.00
52.00	0.00	2.403	440.91	0.73	0.73	0.00
54.00	0.00	2.282	440.73	0.73	0.73	0.00
56.00	0.00	2.161	440.54	0.73	0.73	0.00
58.00	0.00	2.039	440.34	0.73	0.73	0.00
60.00	0.00	1.918	440.14	0.73	0.73	0.00
62.00	0.00	1.797	439.93	0.73	0.73	0.00
64.00	0.00	1.676	439.72	0.69	0.69	0.00
66.00	0.00	1.584	439.55	0.44	0.44	0.00
68.00	0.00	1.525	439.44	0.28	0.28	0.00
70.00	0.00	1.488	439.37	0.18	0.18	0.00
72.00	0.00	1.465	439.33	0.11	0.11	0.00

Stage-Discharge for Pond 19P: Sediment Pond (WSE 450) 72hr Drawdown

Elevation (feet)	Discharge (cfs)	Primary (cfs)	Secondary (cfs)
434.00	0.00	0.00	0.00
434.20	0.00	0.00	0.00
434.40	0.00	0.00	0.00
434.60	0.00	0.00	0.00
434.80	0.00	0.00	0.00
435.00	0.00	0.00	0.00
435.20	0.00	0.00	0.00
435.40	0.00	0.00	0.00
435.60	0.00	0.00	0.00
435.80	0.00	0.00	0.00
436.00	0.00	0.00	0.00
436.20	0.00	0.00	0.00
436.40	0.00	0.00	0.00
436.60	0.00	0.00	0.00
436.80	0.00	0.00	0.00
437.00	0.00	0.00	0.00
437.20	0.00	0.00	0.00
437.40	0.00	0.00	0.00
437.60	0.00	0.00	0.00
437.80	0.00	0.00	0.00
438.00	0.00	0.00	0.00
438.20	0.00	0.00	0.00
438.40	0.00	0.00	0.00
438.60	0.00	0.00	0.00
438.80	0.00	0.00	0.00
439.00	0.00	0.00	0.00
439.20	0.00	0.00	0.00
439.40	0.22	0.22	0.00
439.60	0.51	0.51	0.00
439.80	0.73	0.73	0.00
440.00	0.73	0.73	0.00
440.20	0.73	0.73	0.00
440.40	0.73	0.73	0.00
440.60	0.73	0.73	0.00
440.80	0.73	0.73	0.00
441.00	0.73	0.73	0.00
441.20	0.73	0.73	0.00
441.40	0.82	0.82	0.00
441.60	0.89	0.89	0.00
441.80	0.94	0.94	0.00
442.00	0.98	0.98	0.00
442.20	1.01	1.01	0.00
442.40	1.13	1.13	0.00
442.60	1.23	1.23	0.00
442.80	1.30	1.30	0.00
443.00	1.36	1.36	0.00
443.20	1.42	1.42	0.00
443.40	1.56	1.56	0.00
443.60	1.67	1.67	0.00
443.80	1.77	1.77	0.00
444.00	1.85	1.85	0.00
444.20	1.92	1.92	0.00
444.40	2.08	2.08	0.00
444.60	2.21	2.21	0.00
444.80	2.32	2.32	0.00
445.00	2.41	2.41	0.00
445.20	2.50	2.50	0.00
445.40	2.67	2.67	0.00
445.60	2.82	2.82	0.00
445.80	2.94	2.94	0.00
446.00	3.05	3.05	0.00
446.20	3.16	3.16	0.00
446.40	3.34	3.34	0.00
446.60	3.50	3.50	0.00
446.80	3.64	3.64	0.00
447.00	3.76	3.76	0.00
447.20	3.87	3.87	0.00
447.40	4.07	4.07	0.00
447.60	4.24	4.24	0.00
447.80	4.39	4.39	0.00
448.00	4.52	4.52	0.00
448.20	4.65	4.65	0.00
448.40	4.85	4.85	0.00
448.60	5.04	5.04	0.00
448.80	5.20	5.20	0.00
449.00	5.34	5.34	0.00
449.20	5.48	5.48	0.00
449.40	5.61	5.61	0.00
449.60	5.73	5.73	0.00
449.80	5.85	5.85	0.00
450.00	5.97	5.97	0.00
450.20	10.76	10.76	0.00
450.40	23.04	19.43	3.62
450.60	40.48	27.32	13.16
450.80	54.71	27.54	27.18
451.00	71.47	27.75	43.72
451.20	90.07	27.97	62.10
451.40	**111.04**	**28.19**	**82.86**

Stage-Area-Storage for Pond 19P: Sediment Pond (WSE 450) 72hr Drawdown

Elevation (feet)	Surface (acres)	Storage (acre-feet)	Elevation (feet)	Surface (acres)	Storage (acre-feet)
434.00	0.030	0.000	444.60	1.020	5.509
434.20	0.050	0.008	444.80	1.040	5.715
434.40	0.070	0.020	445.00	1.060	5.925
434.60	0.090	0.036	445.20	1.078	6.139
434.80	0.110	0.056	445.40	1.096	6.356
435.00	0.130	0.080	445.60	1.114	6.577
435.20	0.148	0.108	445.80	1.132	6.802
435.40	0.166	0.139	446.00	1.150	7.030
435.60	0.184	0.174	446.20	1.168	7.262
435.80	0.202	0.213	446.40	1.186	7.497
436.00	0.220	0.255	446.60	1.204	7.736
436.20	0.236	0.301	446.80	1.222	7.979
436.40	0.252	0.349	447.00	1.240	8.225
436.60	0.268	0.401	447.20	1.258	8.475
436.80	0.284	0.457	447.40	1.276	8.728
437.00	0.300	0.515	447.60	1.294	8.985
437.20	0.318	0.577	447.80	1.312	9.246
437.40	0.336	0.642	448.00	1.330	9.510
437.60	0.354	0.711	448.20	1.346	9.778
437.80	0.372	0.784	448.40	1.362	10.048
438.00	0.390	0.860	448.60	1.378	10.322
438.20	0.410	0.940	448.80	1.394	10.600
438.40	0.430	1.024	449.00	1.410	10.880
438.60	0.450	1.112	449.20	1.428	11.164
438.80	0.470	1.204	449.40	1.446	11.451
439.00	0.490	1.300	449.60	1.464	11.742
439.20	0.508	1.400	449.80	1.482	12.037
439.40	0.526	1.503	450.00	1.500	12.335
439.60	0.544	1.610	450.20	1.516	12.637
439.80	0.562	1.721	450.40	1.532	12.941
440.00	0.580	1.835	450.60	1.548	13.249
440.20	0.598	1.953	450.80	1.564	13.561
440.40	0.616	2.074	451.00	1.580	13.875
440.60	0.634	2.199	451.20	1.616	14.195
440.80	0.652	2.328	451.40	**1.652**	**14.521**
441.00	0.670	2.460			
441.20	0.690	2.596			
441.40	0.710	2.736			
441.60	0.730	2.880			
441.80	0.750	3.028			
442.00	0.770	3.180			
442.20	0.790	3.336			
442.40	0.810	3.496			
442.60	0.830	3.660			
442.80	0.850	3.828			
443.00	0.870	4.000			
443.20	0.888	4.176			
443.40	0.906	4.355			
443.60	0.924	4.538			
443.80	0.942	4.725			
444.00	0.960	4.915			
444.20	0.980	5.109			
444.40	1.000	5.307			

ATTACHMENT E

Faircloth Skimmer Sizing Calculator

Calculate Skimmer Size

Basin Volume in Cubic Feet	63,446	Cu.Ft	**Skimmer Size**	**8.0**	Inch
Days to Drain*	1	Days	Orifice Radius	**3.2**	Inch[es]
			Orifice Diameter	**6.4**	Inch[es]

*In NC assume 3 days to drain

Estimate Volume of Basin

	Length	**Width**				
Top of water surface in feet			Feet	**VOLUME**	**0**	**Cu. Ft.**
Bottom dimensions in feet			Feet			
Depth in feet			Feet			

AECOM

Job Scepter: East Landfill | Project No. 60398526 | Page 1 of 1
Description Faircloth skimmer orifice size | Computed by N. Popkowski | Date 12/21/2016
Checked by Bkg | Date 1/6/2017
Sheet ___ of ___
Reference

Determine discharge rate of Faircloth Skimmer

$$Q = CA(2gH)^{0.5}$$

Q = Discharge, cfs
C = Orifice Coefficient = 0.59
A = Area of orifice, sf

$$= \frac{\pi D^2}{4} = \frac{\pi (6.4\,in)^2}{4} = 32.17\,in^2 = 0.22\,sf$$

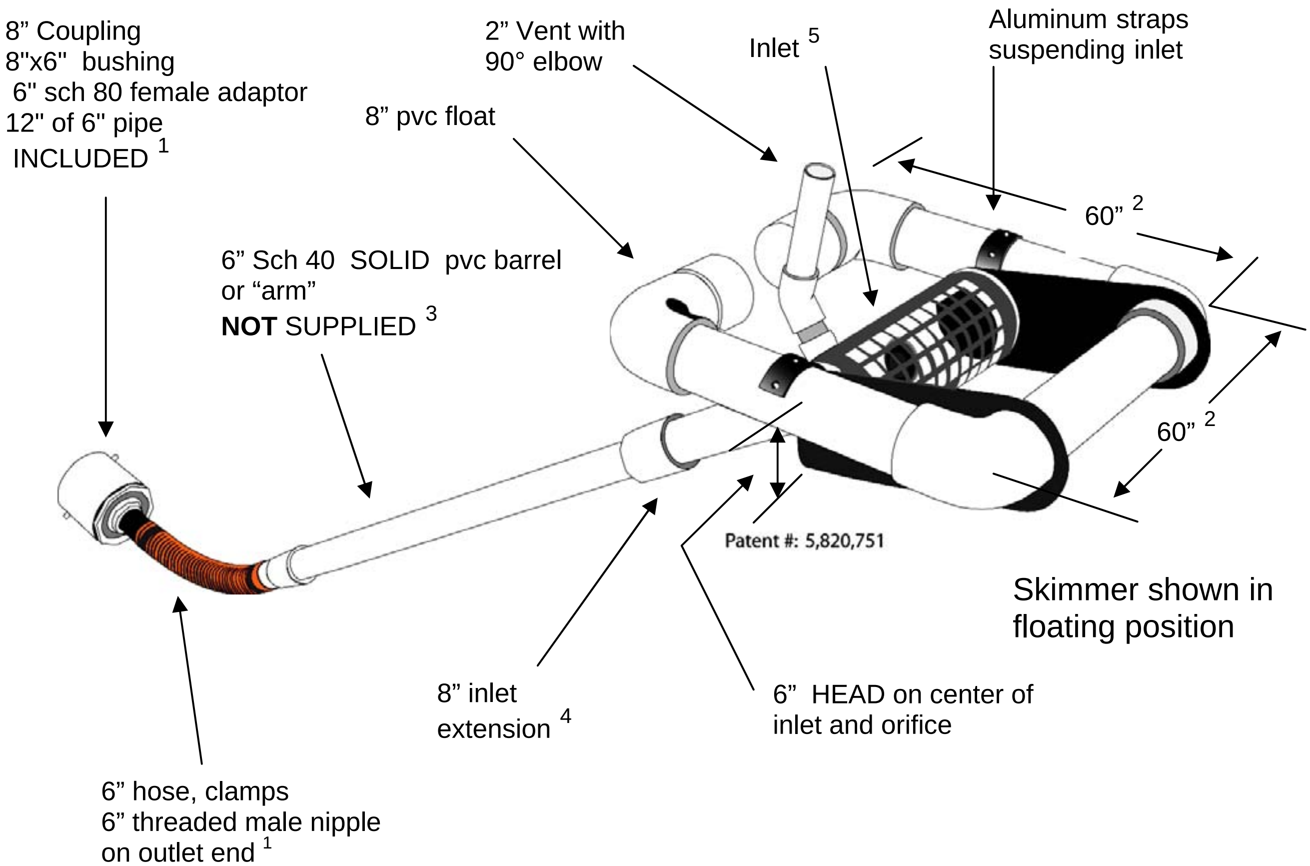
D = orifice Diameter
g = acceleration due to gravity = 32.2 ft/s^2
H = Head above orifice centroid, feet = 0.5 ft

$$Q = 0.59(0.22\,sf)\left[2\left(32.2\,ft/s^2\right)(0.5\,ft)\right]^{0.5}$$

$$\boxed{Q = 0.734\ cfs}$$

8” *Faircloth Skimmer®* Cut Sheet

J. W. Faircloth & Son, Inc.
www.FairclothSkimmer.com



1. Hose can be attached to outlet using the threaded 6” nipple. Typical methods used: on a metal structure a steel stubout welded on the side at the bottom with a 6” threaded coupling or reducers; on a concrete structure with a hole or orifice at the bottom, use a steel plate with a hole and coupling welded to it that will fit over the hole in the concrete and bolted to the structure with sealant.
2. Dimensions are approximate, not intended as plans for construction.
3. Barrel (solid, not foam core pipe) should be 1.4 times the depth of water with a minimum length of 8’ so the inlet can be pulled to the side for maintenance. If more than 12’ long weight may have to be added to inlet to counter the increased buoyancy.
4. Inlet tapers down from 8” maximum inlet to a 6” barrel and hose. Barrel is smaller to reduce buoyancy and tendency to lift inlet but is sufficient for flow through inlet because of slope. The inlet orifice can be reduced using the plug and cutter provided to control the outflow rate.
5. Inlet is 12” pipe between the straps with slots cut in the inlet and aluminum screen door (smaller than shown in illustration) for access to the inlet and orifice inside.
6. Shipped assembled. User glues inlet extension and barrel, installs vent, cuts orifice in plug and attaches to outlet pipe or structure. Includes flexible hose, rope, orifice cutter, etc.
7. Shipped in cardboard box on pallet. Shipping weight 365 lbs, dimension 61"x44"x57".

ATTACHMENT F

Emergency Spillway Lining

AECOM

Job: Scepter: East Landfill	Project No. 60398526	Page 1 of 2	
Description: Channel Lining: Sediment Basin Emergency Spillway Broad-creasted weir (Road Crossing)	Computed by N. Popkowski	Sheet ___ of ___	
	Checked by Bly	Date 12/21/2016	
		Date 1/6/2017	
		Reference	

Reference: HEC-22 Urban Drainage Design Manual Chapter 5

① Determine the energy grade slope

Manning's Equation $Q: \frac{K_a}{n} s^{1/2} AR^{2/3}$

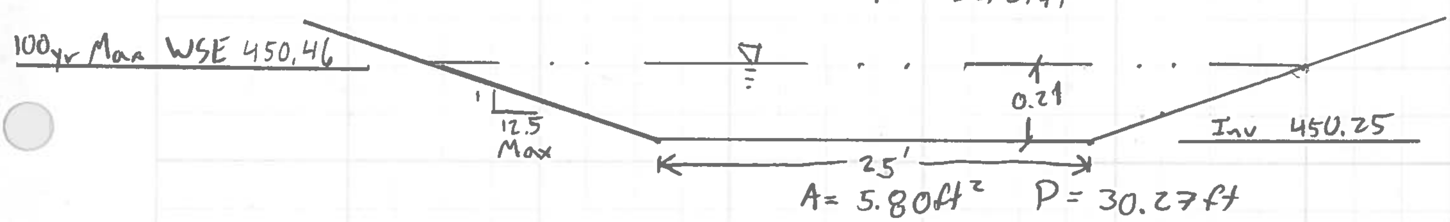
Q = Discharge, ft^3/s = 5.82 cfs → 100 yr storm event assuming primary spillway failure

K_a = 1.49 for US Customary

n = manning's roughness coefficient = 0.035 → riprap/grass

s = energy grade slope, ft/ft

R = hydraulic radius, ft $= A/p = \frac{5.80 ft^2}{30.27 ft} = 0.19 ft$



$$Q = \frac{k_a}{n} s^{1/2} A R^{2/3}$$

$$5.82 cfs = \frac{1.49}{0.035} s^{1/2} (5.80 sf)(0.19 ft)^{2/3}$$

$S = 0.005$ ft/ft (ENERGY GRADE SLOPE ACROSS S/WAY)

② Check Velocity

$Q = vA$

$5.82 cfs = v \; 5.80 sf$

$v = 1.00 fps$

Q: Discharge, ft^3/s = 5.82 cfs

A: Area, sf = 5.80 sf

AECOM

Job: Scepter : East Landfill — Project No. 60398526 — Sheet ___ of ___

Description: Channel Lining : Sediment Basin Emergency Spillway Broad-creasted weir (Road Crossing)

Computed by: N. Popkowski — Date: 12/21/2016

Checked by: Blz — Date: 1/6/2017

Reference

③ Determine Channel Protection Needed

Compute the maximum shear stress

$\tau_d = \gamma d s$

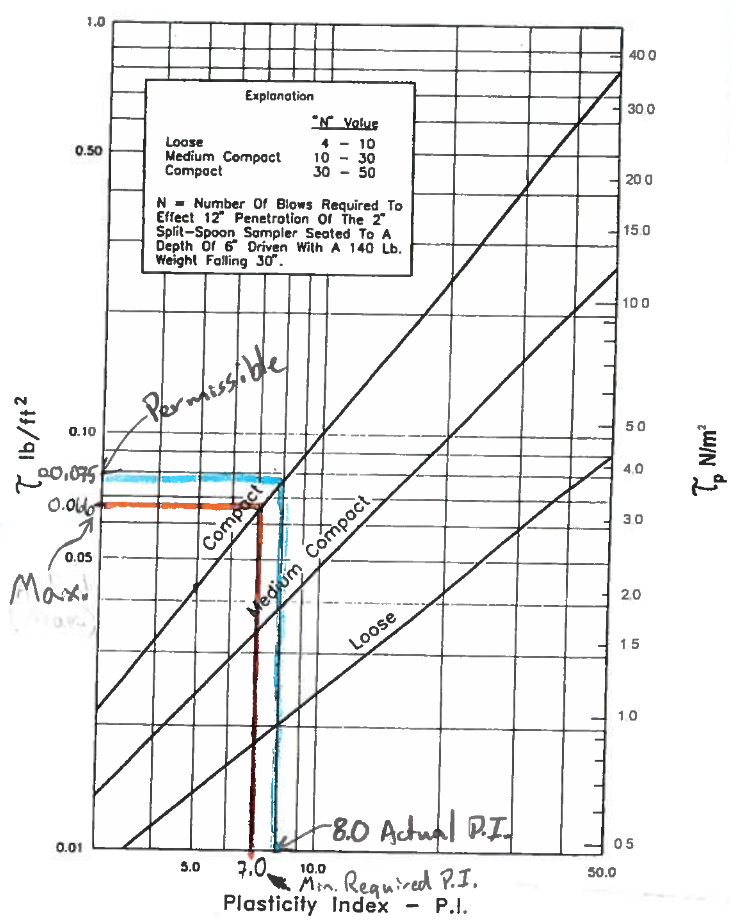
τ_d = maximum shear stress, lb/ft²
d = maximum depth of flow, ft = 0.21 ft
s = average energy slope, ft/ft = 0.005 ft/ft
γ = unit weight of water, lb/ft³ = 62.4 lb/ft³

$\tau_d = 62.4\ \text{lb/ft}^3 (0.21\ \text{ft})(0.005)$
$\tau_d = 0.066\ \text{lb/ft}^3$

④ Select Channel Lining

- Road Crossing Surface shall be constructed with on-site Chert.
- Minimum Plasticity Index for Bare Cohesive Soil = 7.0 (see Chart 24, right) Reference HEC-22
- Plasticity Index of on-site Chert = 8.0 (see attached boring log)

CHART 24



Permissible Shear Stress For Cohesive Soils.

⑤ Assure Permissible Shear Stress is greater than maximum shear stress

- Permissible Shear Stress = 0.075
- Maximum Shear Stress = 0.066

Permissible > Maximum

Onsite Chert is <u>Adequate</u> at Sediment Basin Emergency Spillway Road Crossing

McCray Drilling, LLC.
Geotechnical & Environmental
Drilling & Testing Services
6756 Buckles Cove • Memphis, TN • 38133 • P. 901.385.1199 • F. 901.386.6614

SUMMARY OF LABORATORY RESULTS

CLIENT Scepter, Inc.
PROJECT NAME Scepter Landfill
PROJECT NUMBER A-2-726
PROJECT LOCATION Waverly, TN

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	58.5	30	21	9	19	46	SC	17.5			
B-1	70.0	26	19	7	12.5	63	CL-ML	18.0			
B-1	75.0	26	17	9	12.5	58	CL	17.0			
B-4	8.5		NP	NP	19	27	SM	15.3			
B-4	20.0	25	17	8	12.5	31	SC	15.3			
B-4	33.5	33	19	14	12.5	55	CL	24.8			
B-5	13.5		NP	NP	12.5	66	ML	18.1			
B-5	20.0	25	17	8	12.5	50	CL	17.1			
B-5	28.5		NP	NP	25	31	SM	16.1			
MW-4	28.5	40	32	8	12.5	56	ML	33.0			
MW-E1	23.5	29	17	12							
MW-E1	28.5	39	24	15	12.5	73	CL	31.0			
MW-E1	58.5	40	31	9	19	67	ML	31.4			
MW-E1	80.0	30	29	1	12.5	53	ML	15.7			
MW-E1	88.5	31	19	12	19	37	SC	31.2			
MW-E2	8.5	38	25	13	19	43	SM	36.3			
MW-E2	23.5	31	22	9	19	25	GC	39.3			
MW-E2	25.0	31	24	7	19	48	SM	39.8			
MW-E2	28.0	35	28	7	12.5	37	SM	38.4			
MW-E2	33.5	35	27	8	19	36	SM	33.7			
PZ-2	13.5	27	18	9	19	41	SC	14.7			
PZ-2	18.5	31	24	7	19	40	SM	18.8			
PZ-2	48.5	32	25	7	25	46	SM	11.9			
PZ-2	50.0	27	19	8	12.5	41	SC	13.8			
PZ-3	28.5		NP	NP	19	14	SM				
PZ-3	38.5		NP	NP	12.5	49	SM	6.1			
PZ-3	45.0	27	19	8	19	50	CL	13.8			
PZ-3	47.0	33	28	5	9.5	68	ML				
PZ-3	48.5	35	22	13	19	62	CL	25.5			
PZ-3	60.0		NP	NP	12.5	50	ML				
PZ-3	73.5	32	27	5	19	39	SM	37.2			
PZ-6	3.5	38	27	11	12.5	44	SM	35.9			
PZ-6	7.5		NP	NP	9.5	47	SM	30.7			
PZ-6	15.0	35	23	12	19	48	SC	25.2			
PZ-6	28.5	33	28	5	25	30	SM	32.9			
PZ-6	38.5		NP	NP	19	33	SM	40.9			

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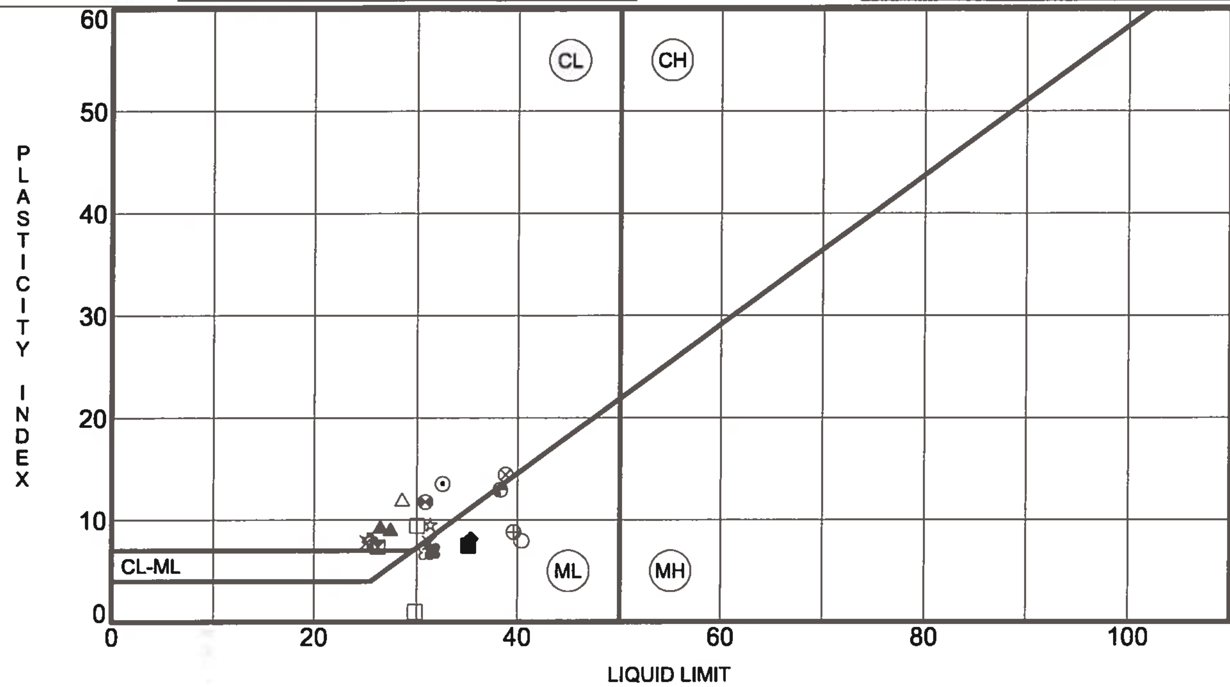
ATTERBERG LIMITS' RESULTS
ASTM D4318

CLIENT Scepter, Inc. **PROJECT NAME** Scepter Landfill

PROJECT NUMBER A-2-726 **PROJECT LOCATION** Waverly, TN



	Specimen Identification		LL	PL	PI	Fines	Classification
□	B-1	58.5	30	21	9	46	Red Silty Clayey Sand with Gravel (SC)
⊠	B-1	70.0	26	19	7	63	Brown Sandy Clayey Silt with Trace of Gravel (CL-ML)
▲	B-1	75.0	26	17	9	58	Brown Sandy Silty Clay with Trace of Gravel (CL)
×	B-4	20.0	25	17	8	31	Brown Clayey Silty Sand with Gravel (SC)
⊙	B-4	33.5	33	19	14	55	Brown Sandy Silty Clay with Trace of Gravel (CL)
✪	B-5	20.0	25	17	8	50	Brown Sandy Clayey Silt with Trace of Gravel (CL)
○	MW-4	28.5	40	32	8	56	Tan and Red Sandy Clayey Silt with Trace of Gravel (ML)
△	MW-E1	23.5	29	17	12		Reddish-Gray Sandy Clay with Gravel
⊗	MW-E1	28.5	39	24	15	73	Tan Sandy Silty Clay with Trace of Gravel (CL)
⊕	MW-E1	58.5	40	31	9	67	Tan Clayey Sandy Silt with Trace of Gravel (ML)
□	MW-E1	80.0	30	29	1	53	Red and Brown Sandy Clayey Silt with Trace of Gravel (ML)
⊖	MW-E1	88.5	31	19	12	37	Red and Brown Clayey Silty Sand with Gravel (SC)
◕	MW-E2	8.5	38	25	13	43	Tan Clayey Silty Sand with Gravel (SM)
☆	MW-E2	23.5	31	22	9	25	Brown Clayey Silty Sandy Gravel (GC)
⌘	MW-E2	25.0	31	24	7	48	Brown and Gray Clayey Silty Sand with Gravel (SM)
■	MW-E2	28.0	35	28	7	37	Tan Clayey Silty Sand with Gravel (SM)
◆	MW-E2	33.5	35	27	8	36	Tan Clayey Silty Sand with Trace of Gravel (SM)
▲	PZ-2	13.5	27	18	9	41	Reddish Brown Clayey Sandy Silt with Gravel (SC)
×	PZ-2	18.5	31	24	7	40	Brown Clayey Silty Sand with Gravel (SM)
✖	PZ-2	48.5	32	25	7	46	Tan Clayey Silty Sand with Gravel (SM)

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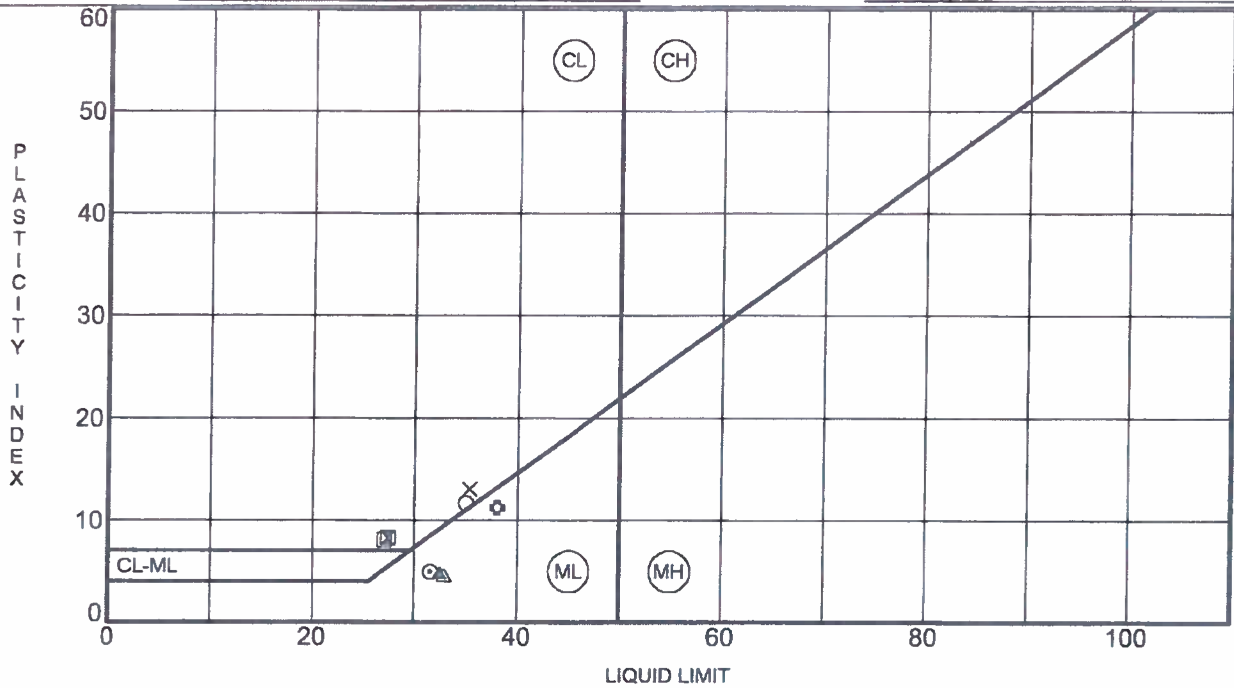
ATTERBERG LIMITS' RESULTS
ASTM D4318

CLIENT Scepter, Inc.
PROJECT NAME Scepter Landfill
PROJECT NUMBER A-2-726
PROJECT LOCATION Waverly, TN



	Specimen Identification		LL	PL	PI	Fines	Classification
□	PZ-2	50.0	27	19	8	41	Brown Clayey Silty Sand with Trace of Gravel (SC)
⊠	PZ-3	45.0	27	19	8	50	Tan Sandy Silty Clay with Gravel (CL)
▲	PZ-3	47.0	33	28	5	68	Tan and Red Sandy Silt (ML)
×	PZ-3	48.5	35	22	13	62	Tan Sandy Clayey Silt with Gravel (CL)
⊙	PZ-3	73.5	32	27	5	39	Red Clayey Silty Sand with Gravel (SM)
✪	PZ-6	3.5	38	27	11	44	Reddish Brown Clayey Silty Sand with Gravel (SM)
○	PZ-6	15.0	35	23	12	48	Brown Clayey Silty Sand with Gravel (SC)
△	PZ-6	28.5	33	28	5	30	Red and Brown Clayey Silty Sand with Gravel (SM)

ATTERBERG LIMITS T:\PROJECTS\MCCRAY DRILLING\A-2-726 SCEPTER SCEPTER LANDFILL.GPJ 4/13/16

Hydraulic Analysis Report

Project Data

Project Title:

Designer:

Project Date: Wednesday, December 21, 2016

Project Units: U.S. Customary Units

Notes:

Channel Analysis: Emergency Spillway - Road Crossing

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 16.0000 ft/ft

Side Slope 2 (Z2): 16.0000 ft/ft

Channel Width: 25.0000 ft

Longitudinal Slope: 0.0050 ft/ft

Manning's n: 0.0350

Flow: 5.8200 cfs

Result Parameters

Depth: 0.2090 ft

Area of Flow: 5.9232 ft^2

Wetted Perimeter: 31.7003 ft

Hydraulic Radius: 0.1868 ft

Average Velocity: 0.9826 ft/s

Top Width: 31.6873 ft

Froude Number: 0.4005

Critical Depth: 0.1159 ft

Critical Velocity: 1.8692 ft/s

Critical Slope: 0.0375 ft/ft

Critical Top Width: 28.71 ft

Calculated Max Shear Stress: 0.0652 lb/ft^2

Calculated Avg Shear Stress: 0.0583 lb/ft^2

Hydraulic Analysis Report

Project Data

Project Title:

Designer:

Project Date: Wednesday, December 21, 2016

Project Units: U.S. Customary Units

Notes:

Channel Analysis: Emergency Spillway - Backslope

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 16.0000 ft/ft

Side Slope 2 (Z2): 16.0000 ft/ft

Channel Width: 25.0000 ft

Longitudinal Slope: 0.3300 ft/ft

Manning's n: 0.0350

Flow: 5.8200 cfs

Result Parameters

Depth: 0.0606 ft

Area of Flow: 1.5740 ft^2

Wetted Perimeter: 26.9433 ft

Hydraulic Radius: 0.0584 ft

Average Velocity: 3.6975 ft/s

Top Width: 26.9395 ft

Froude Number: 2.6957

Critical Depth: 0.1160 ft

Critical Velocity: 1.8682 ft/s

Critical Slope: 0.0374 ft/ft

Critical Top Width: 28.71 ft

Calculated Max Shear Stress: 1.2481 lb/ft^2

Calculated Avg Shear Stress: 1.2030 lb/ft^2

APPENDIX F.1B

Culvert Design (Rev 3)

I. PURPOSE

The purpose of this analysis is to design the proposed culverts associated with Scepter's Class II Disposal Facility in Waverly, TN consistent with state regulations.

II. SITE AND PROJECT DESCRIPTION

The culvert's design was performed as part of the Part II Solid Waste Permit Application. The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management. Refer to Exhibit 1 for a depiction of the culverts locations in relation to the landfill.

The following sections summarize the design criteria, procedure, assumptions, and results of the culvert's design.

III. REGULATORY REQUIREMENTS / DESIGN CRITERIA

The below parts of the TDEC regulations specify requirements for the design of the culverts:

Rule 0400-11-01-.04(2)(i)1.

The operator must design, construct, operate, and maintain a run-on control system capable of preventing flow onto the active portion of the facility for all flow up to and including peak discharge from a 24-hour, 25-year storm.

Rule 0400-11-01-.04(2)(i)2.

The operator must design, construct, operate, and maintain a run-off management system to collect and control at least the peak flow resulting from a 24-hour, 25 year storm.

The results of the analysis presented herein show that the culverts are designed to control and convey at least the peak flow resulting from a 25-year/24-hour storm.

IV. PROCEDURE

Design of the landfill site stormwater features was an iterative process beginning with basic assumptions and a proposed grading plan for the site. The hydraulic features of the culverts were initially assumed and then confirmed through multiple iterations.

The AutoCAD Civil 3D software package was used to generate the proposed site grading plan and subsequently to determine drainage areas, volumes, and other site geometry. HydroCAD (version 10.00) modeling software was used to conduct the hydrologic calculations and the Federal Highway

Administation's HY-8 (version 7.2) was used to conduct hydraulic calculations for this analysis with inputs based on the site geometry, rainfall data, and other design assumptions.

The HydroCAD model was used to generate peak flow rates for the upstream watershed features of the culvert. HY-8 was used to generate peak headwater and conveyance capacities of each given culvert. The number, size, and elevation of culverts were designed/determined such that the regulatory requirements would be met while minimizing cost through reduced number and size of structures to the extent possible. The proposed culverts will pass flows equal to and lesser than those generated by the 24-hour, 25-year storm.

To provide energy dissipation and greatly reduce potential for erosion downstream of the culvert outlets, riprap aprons designed in accordance with the TDEC Erosion and Sediment Control Handbook are provided for the 24-hour, 25-year design storm peak discharge (Appendix F1.G).

V. NOTES/ASSUMPTIONS

The following is a list of key notes and assumptions made in completing this analysis.

- The rainfall intensities for Waverly, Tennessee are as follows and were obtained from NOAA.gov and are included as Attachment A.:
  - 6.39 inches to represent the 24-hour, 25-year storm
  - 8.12 inches to represent the 24-hour, 100-year storm
- Within the HydroCAD program, the runoff was calculated using the SCS TR-20 method.
- Runoff curve numbers (CN) used in the analysis were as follows:
  - 78 for landfill vegetated final cap cover and offsite areas,
  - 98 for sediment basin water surface.
  - 77 for exposed and disturbed soil (daily cover); however exposed and disturbed soil was ignored to be conservative and 78 was used within landfill construction footprint.
- The time of concentration was calculated using the Curve Number Method in HydroCAD which takes inputs for each drainage area of the longest hydraulic flow path and average land slope.
- Pipe flow in HydroCAD was calculated by the program using Manning's equation for low flow conditions and the orifice equation for high flow (submerged) conditions.
- Flow into the catch basins in HydroCAD was calculated by the program using weir flow for low flow conditions and the orifice equation for high flow (submerged) conditions.

VI. SUMMARY OF RESULTS

A summary of all culverts for the facility is presented in Exhibit 1. The results of the culverts design show the TDEC regulations under the Solid Waste requirements have been met (see also Attachments C, D, E, F, and G for associated HydroCAD output reports). As Table 1, below, illustrates, the structures properly control and convey the water volume generated by the 24-hour, 25-year storm such that the peak water surface elevation immediately upstream stays safely below the impounding sideslopes.

Table 1 - Culvert Summary

Culvert ID	Description	Peak Discharge (cfs)	Barrels/Size/Material	Control	Required Head (ft)	Potential Head (ft)	Velocity (fps)
C1	South Access Road	21.90	1 - 18” RCP	Inlet	7.23	38.00	14.84
C2	North Access Road	36.87	1 - 30” RCP	Inlet	3.88	26.68	12.15
C3	Temporary Sediment Basin	42.34	2 - 30” RCP	Inlet	2.39	5.55	10.62
C4	North Sediment Basin	83.65	3 – 30” CPP	Inlet	2.92	4.5	11.41
C5	South Sediment Basin	58.35	2 – 30” CPP	Inlet	3.04	4.5	11.51

VII. CONCLUSIONS

The proposed grading of the landfill in combination with the design of the culvert is sufficient to safely control and convey the 24-hour, 25-year storm as stipulated by TDEC regulations. Appropriate outlet protection riprap for each culvert based on regulatory guidance is incorporated (Appendix F1.G). Calculations for proper conveyance of upstream and downstream flows were not presented in this analysis and will be performed separately. Refer to accompanying calculations for upstream conveyance design features (storm water terraces and perimeter channels) entering the culverts and the sediment basin.

VIII. ATTACHMENTS

Figures:

Exhibit 1: Drainage Map

Attachments:

Attachment A: NOAA Precipitation Frequency Data
Attachment B: NRCS Hydrologic Soil Group
Attachment C: South Sed. Basin Culvert: Drainage Map, HydroCAD report, and HY-8 Report
Attachment D: North Sed. Basin Culvert: Drainage Map, HydroCAD report, and HY-8 Report
Attachment E: Temporary Sed. Basin Culvert: Drainage Map, HydroCAD report, and HY-8 Report
Attachment F: South Access Road Culvert: Drainage Map, HydroCAD report, and HY-8 Report
Attachment G: North Access Road Culvert: Drainage Map, HydroCAD report, and HY-8 Report

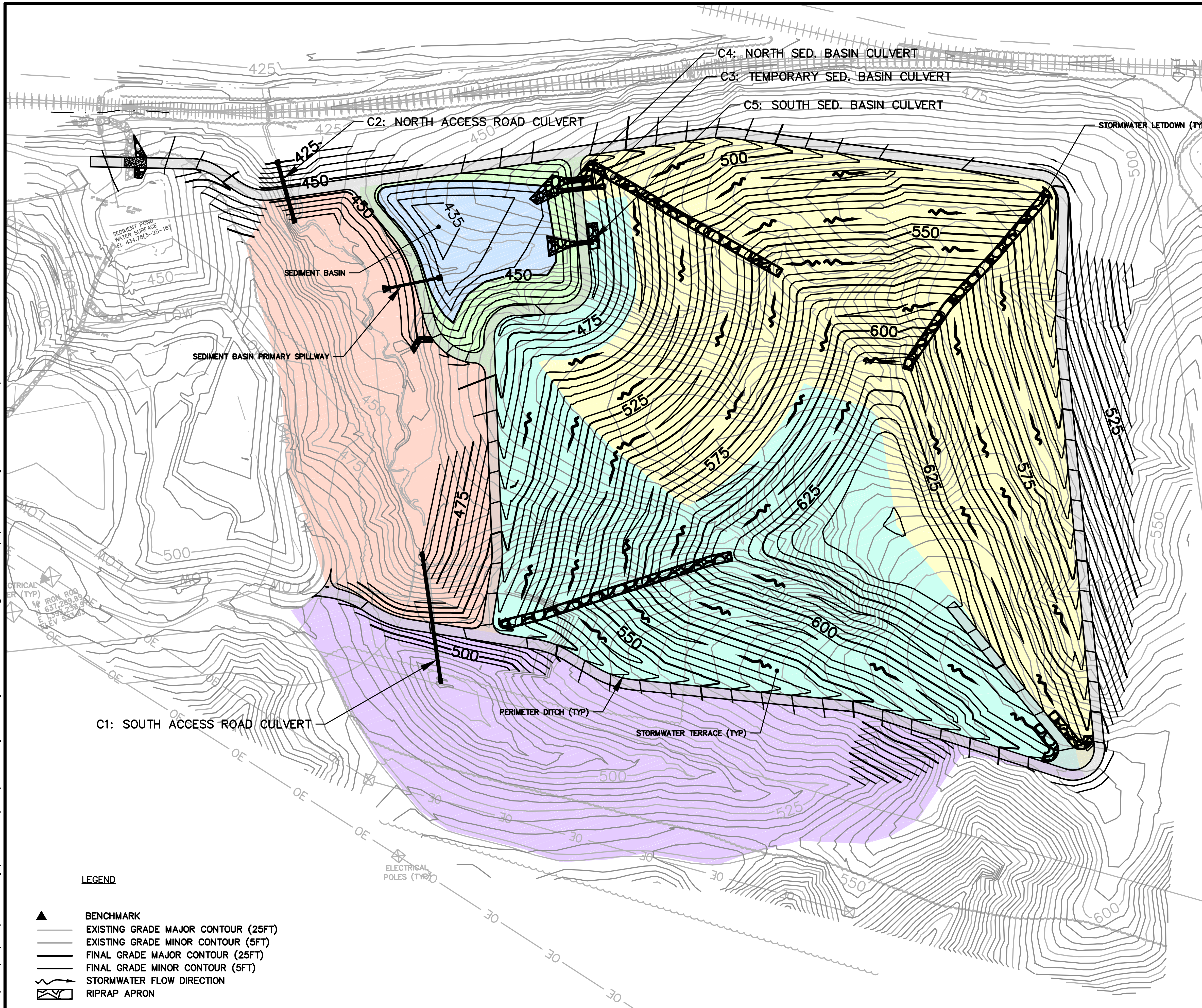
IX. REFERENCES

1. TDEC, *"Rules of Tennessee Department of Environment and Conservation, Chapter 0400-11-01 – Solid Waste Processing and Disposal"*, Solid Waste Management, September 2012.

2. TDEC, *"Erosion and Sediment Control Handbook"*, Fourth Edition, August 2012.

EXHIBIT 1

Drainage Map



DRAINAGE AREAS

NORTH SED. BASIN CULVERT AREA = 15.99 ACRES CN = 78 TC = 14.0 MINS
SOUTH SED. BASIN CULVERT AREA = 12.06 ACRES CN = 78 TC = 16.5 MINS
SED. BASIN OPEN WATER AREA = 1.50 ACRES CN = 98 TC = 5.0 MINS
SED. BASIN SOIL AREA = 1.50 ACRES CN = 78 TC = 5.0 MINS
NORTH ACCESS ROAD CULVERT AREA = 7.260 CN = 49 TC = 6.0 MINS
SOUTH ACCESS ROAD CULVERT AREA-1 = 9.24 ACRES; CN-1 = 49 AREA-2 = 1 ACRE; CN-2 = 77 TC = 9.9 MINS

AECOM

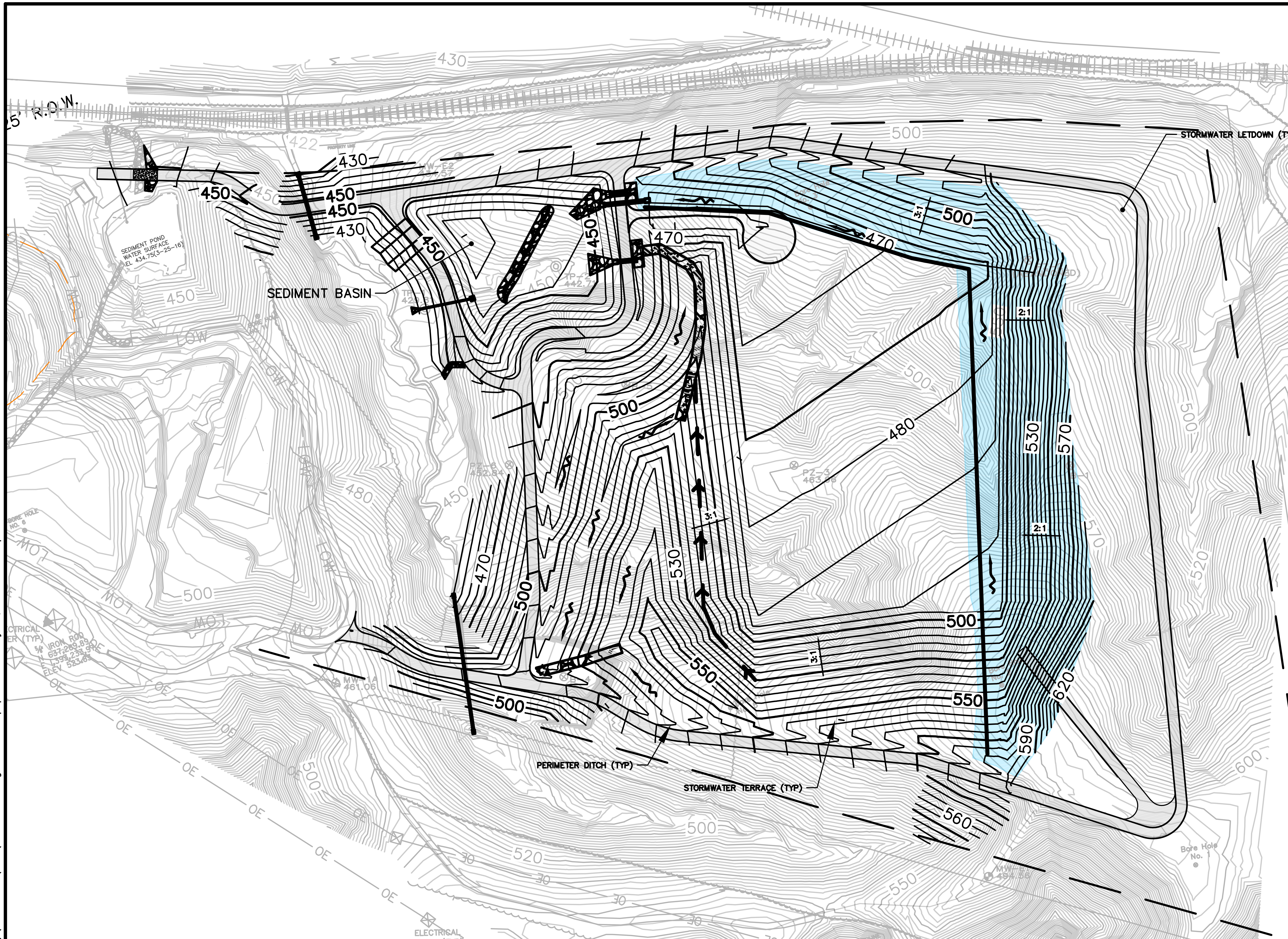
SCEPTER, INC.

WAVERLY, TENNESSEE

ROAD CULVERT DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/4/17	1A

S:\2016\Scepter\028 Waverly\02 Landfill\CAD\x-tributary boundary-sediment basin.dwg User: nick.popkowski May 04, 2017 - 2:04pm



NORTH - NAD83

0 100' 200'

GRAPHIC SCALE

DRAINAGE AREAS

TEMPORARY SED. BASIN CULVERT
AREA = 6.51 ACRES
CN = 78
TC = 7.6 MINS

LEGEND

- ▲ BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (25FT)
- EXISTING GRADE MINOR CONTOUR (5FT)
- FINAL GRADE MAJOR CONTOUR (25FT)
- FINAL GRADE MINOR CONTOUR (5FT)
- STORMWATER FLOW DIRECTION

AECOM

SCEPTER, INC.
WAVERLY, TENNESSEE

TEMPORARY SEDIMENT BASIN CULVERT DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/4/17	1B

S:\2016\Scepter\028 Waverly\02 Landfill\CAD\o-Phase 2.dwg User: nick.popkowski May 04, 2017 - 2:06pm

ATTACHMENT A

NOAA Precipitation Frequency Data

NOAA Atlas 14, Volume 2, Version 3
Location name: Waverly, Tennessee, US*
Latitude: 36.0689°, Longitude: -87.9462°
Elevation: 523 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF tabular | PF graphical | Maps & aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.405 (0.368-0.450)	0.476 (0.434-0.529)	0.547 (0.497-0.607)	0.604 (0.548-0.668)	0.675 (0.610-0.746)	0.728 (0.655-0.803)	0.781 (0.699-0.862)	0.832 (0.742-0.917)	0.898 (0.794-0.990)	0.948 (0.833-1.05)
10-min	0.647 (0.588-0.718)	0.761 (0.694-0.845)	0.876 (0.796-0.972)	0.965 (0.877-1.07)	1.08 (0.972-1.19)	1.16 (1.04-1.28)	1.24 (1.11-1.37)	1.32 (1.18-1.45)	1.42 (1.26-1.57)	1.49 (1.31-1.65)
15-min	0.809 (0.735-0.898)	0.957 (0.872-1.06)	1.11 (1.01-1.23)	1.22 (1.11-1.35)	1.36 (1.23-1.51)	1.47 (1.32-1.62)	1.57 (1.40-1.73)	1.66 (1.48-1.83)	1.79 (1.58-1.97)	1.88 (1.65-2.07)
30-min	1.11 (1.01-1.23)	1.32 (1.20-1.47)	1.58 (1.43-1.75)	1.77 (1.61-1.96)	2.02 (1.83-2.23)	2.21 (1.99-2.44)	2.40 (2.15-2.65)	2.59 (2.31-2.86)	2.84 (2.51-3.14)	3.04 (2.67-3.35)
60-min	1.38 (1.26-1.54)	1.66 (1.51-1.84)	2.02 (1.83-2.24)	2.30 (2.09-2.55)	2.69 (2.43-2.97)	2.99 (2.70-3.31)	3.31 (2.96-3.65)	3.63 (3.24-4.01)	4.08 (3.61-4.50)	4.43 (3.89-4.89)
2-hr	1.59 (1.44-1.76)	1.91 (1.73-2.11)	2.33 (2.11-2.59)	2.69 (2.43-2.98)	3.19 (2.87-3.52)	3.60 (3.22-3.98)	4.03 (3.59-4.46)	4.49 (3.98-4.97)	5.14 (4.52-5.70)	5.67 (4.95-6.29)
3-hr	1.72 (1.57-1.89)	2.05 (1.87-2.27)	2.52 (2.29-2.78)	2.91 (2.64-3.20)	3.45 (3.12-3.80)	3.91 (3.52-4.30)	4.39 (3.93-4.82)	4.90 (4.37-5.39)	5.64 (4.98-6.20)	6.24 (5.47-6.88)
6-hr	2.13 (1.93-2.35)	2.53 (2.30-2.80)	3.10 (2.82-3.43)	3.58 (3.25-3.95)	4.26 (3.84-4.69)	4.83 (4.33-5.32)	5.43 (4.85-5.98)	6.09 (5.40-6.70)	7.01 (6.17-7.72)	7.78 (6.78-8.58)
12-hr	2.58 (2.36-2.85)	3.08 (2.82-3.40)	3.79 (3.46-4.18)	4.38 (3.99-4.82)	5.22 (4.73-5.74)	5.92 (5.34-6.51)	6.66 (5.97-7.32)	7.46 (6.65-8.20)	8.60 (7.59-9.45)	9.54 (8.36-10.5)
24-hr	3.14 (2.88-3.44)	3.76 (3.45-4.12)	4.64 (4.25-5.09)	5.37 (4.91-5.87)	6.39 (5.82-6.98)	7.23 (6.56-7.89)	8.12 (7.32-8.85)	9.06 (8.12-9.88)	10.4 (9.22-11.3)	11.5 (10.1-12.5)
2-day	3.75 (3.46-4.10)	4.49 (4.14-4.92)	5.53 (5.09-6.04)	6.36 (5.84-6.93)	7.52 (6.89-8.19)	8.46 (7.72-9.21)	9.45 (8.56-10.3)	10.5 (9.44-11.4)	11.9 (10.6-13.0)	13.1 (11.6-14.3)
3-day	4.01 (3.69-4.37)	4.80 (4.42-5.24)	5.89 (5.43-6.43)	6.77 (6.22-7.37)	7.98 (7.31-8.69)	8.96 (8.17-9.75)	9.97 (9.04-10.9)	11.0 (9.94-12.0)	12.5 (11.2-13.7)	13.6 (12.1-15.0)
4-day	4.26 (3.93-4.64)	5.11 (4.71-5.56)	6.26 (5.77-6.82)	7.18 (6.61-7.81)	8.45 (7.74-9.18)	9.46 (8.62-10.3)	10.5 (9.52-11.4)	11.6 (10.4-12.6)	13.0 (11.7-14.3)	14.2 (12.6-15.6)
7-day	5.06 (4.65-5.50)	6.06 (5.58-6.59)	7.43 (6.83-8.07)	8.52 (7.81-9.25)	10.0 (9.17-10.9)	11.3 (10.2-12.2)	12.6 (11.4-13.6)	13.9 (12.5-15.1)	15.7 (14.0-17.2)	17.2 (15.2-18.8)
10-day	5.77 (5.34-6.22)	6.90 (6.38-7.45)	8.39 (7.75-9.05)	9.55 (8.81-10.3)	11.1 (10.2-12.0)	12.4 (11.3-13.3)	13.6 (12.4-14.7)	14.9 (13.5-16.1)	16.6 (15.0-18.0)	18.0 (16.1-19.6)
20-day	7.88 (7.35-8.46)	9.36 (8.74-10.1)	11.1 (10.4-12.0)	12.5 (11.6-13.4)	14.2 (13.2-15.3)	15.5 (14.4-16.7)	16.8 (15.5-18.1)	18.1 (16.6-19.5)	19.7 (18.0-21.2)	20.9 (19.0-22.6)
30-day	9.63 (9.00-10.3)	11.4 (10.7-12.2)	13.4 (12.6-14.4)	15.0 (14.0-16.0)	16.9 (15.8-18.1)	18.4 (17.1-19.7)	19.9 (18.5-21.3)	21.4 (19.7-22.9)	23.2 (21.3-24.9)	24.6 (22.5-26.4)
45-day	12.1 (11.3-12.9)	14.3 (13.4-15.2)	16.7 (15.7-17.8)	18.5 (17.3-19.7)	20.8 (19.4-22.1)	22.4 (20.9-23.9)	24.1 (22.4-25.6)	25.6 (23.7-27.3)	27.6 (25.4-29.5)	29.0 (26.6-31.0)
60-day	14.5 (13.6-15.4)	17.1 (16.0-18.2)	19.8 (18.6-21.1)	21.8 (20.4-23.2)	24.3 (22.7-25.9)	26.1 (24.4-27.9)	27.8 (25.9-29.7)	29.5 (27.4-31.5)	31.4 (29.1-33.6)	32.9 (30.3-35.2)

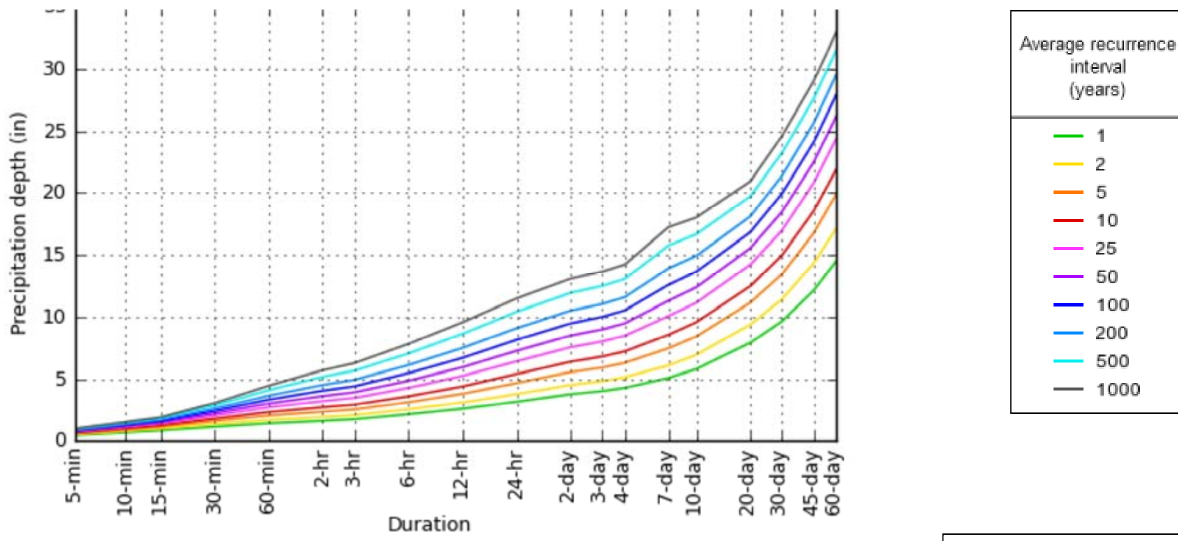
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

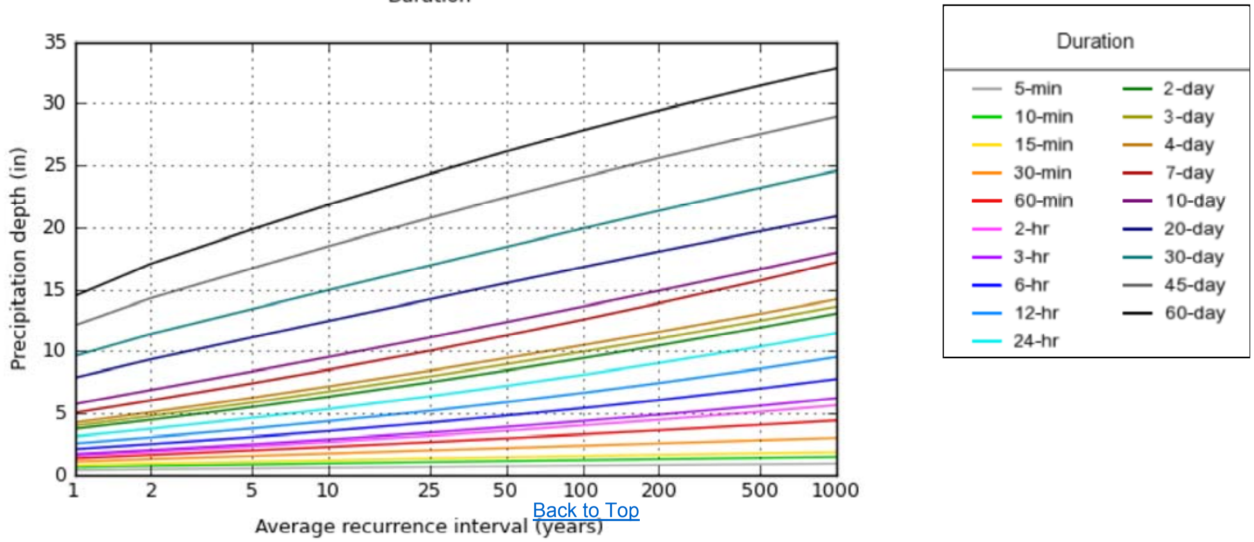
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical





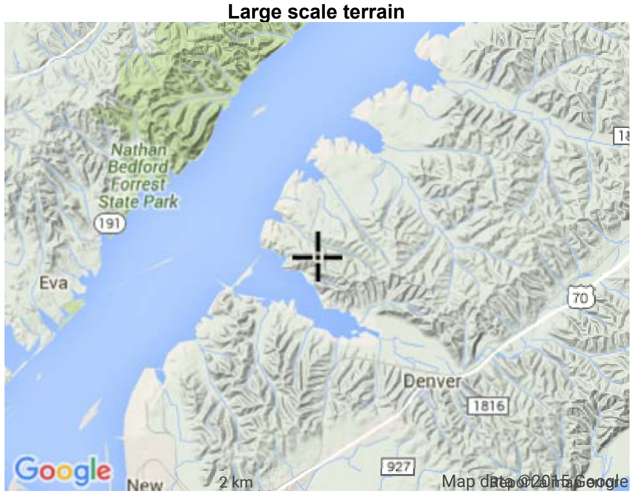
[Back to Top](#)

NOAA Atlas 14, Volume 2, Version 3 Created (GMT): Tue Dec 1 22:54:15 2015

Maps & aerials

Small scale terrain

Large scale terrain



Large scale map

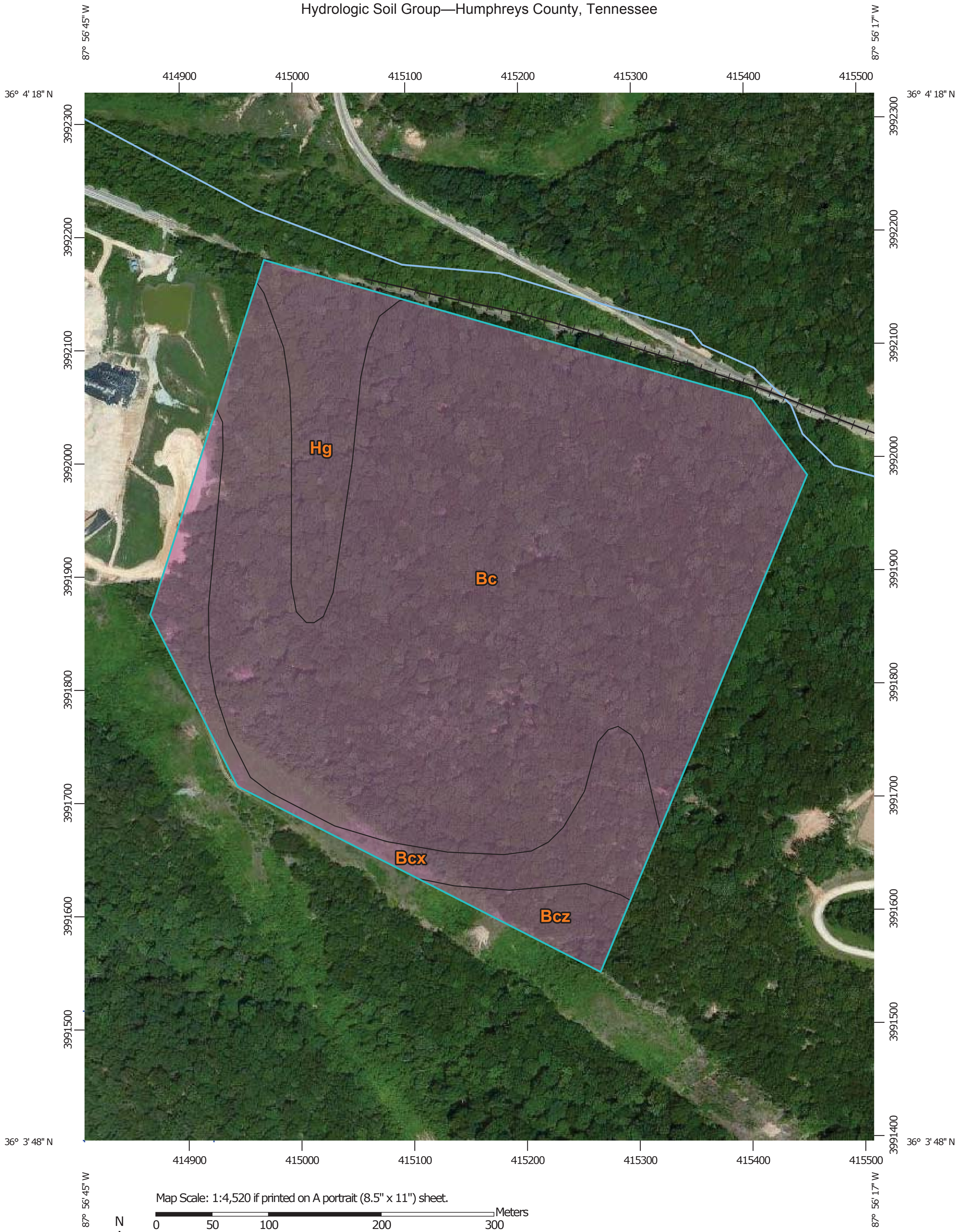
Large scale aerial

[Back to Top]

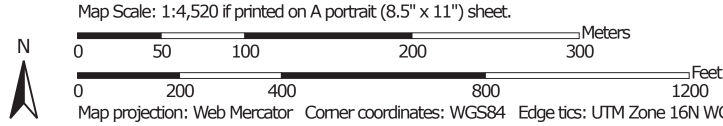
ATTACHMENT B

NRCS Hydrologic Soil Group

Hydrologic Soil Group—Humphreys County, Tennessee



Map Scale: 1:4,520 if printed on A portrait (8.5" x 11") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features

Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Humphreys County, Tennessee
Survey Area Data: Version 10, Sep 28, 2015

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Mar 17, 2011—May 30, 2011

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — Humphreys County, Tennessee (TN085)				
Map unit symbol	**Map unit name**	**Rating**	**Acres in AOI**	**Percent of AOI**
Bc	Bodine cherty silt loam	A	45.2	78.8%
Bcx	Bodine cherty silt loam, slope	A	6.0	10.5%
Bcz	Bodine cherty silt loam, steep	A	1.6	2.8%
Hg	Humphreys gravelly silt loam, 2 to 5 percent slopes	A	4.5	7.9%
Totals for Area of Interest			**57.4**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

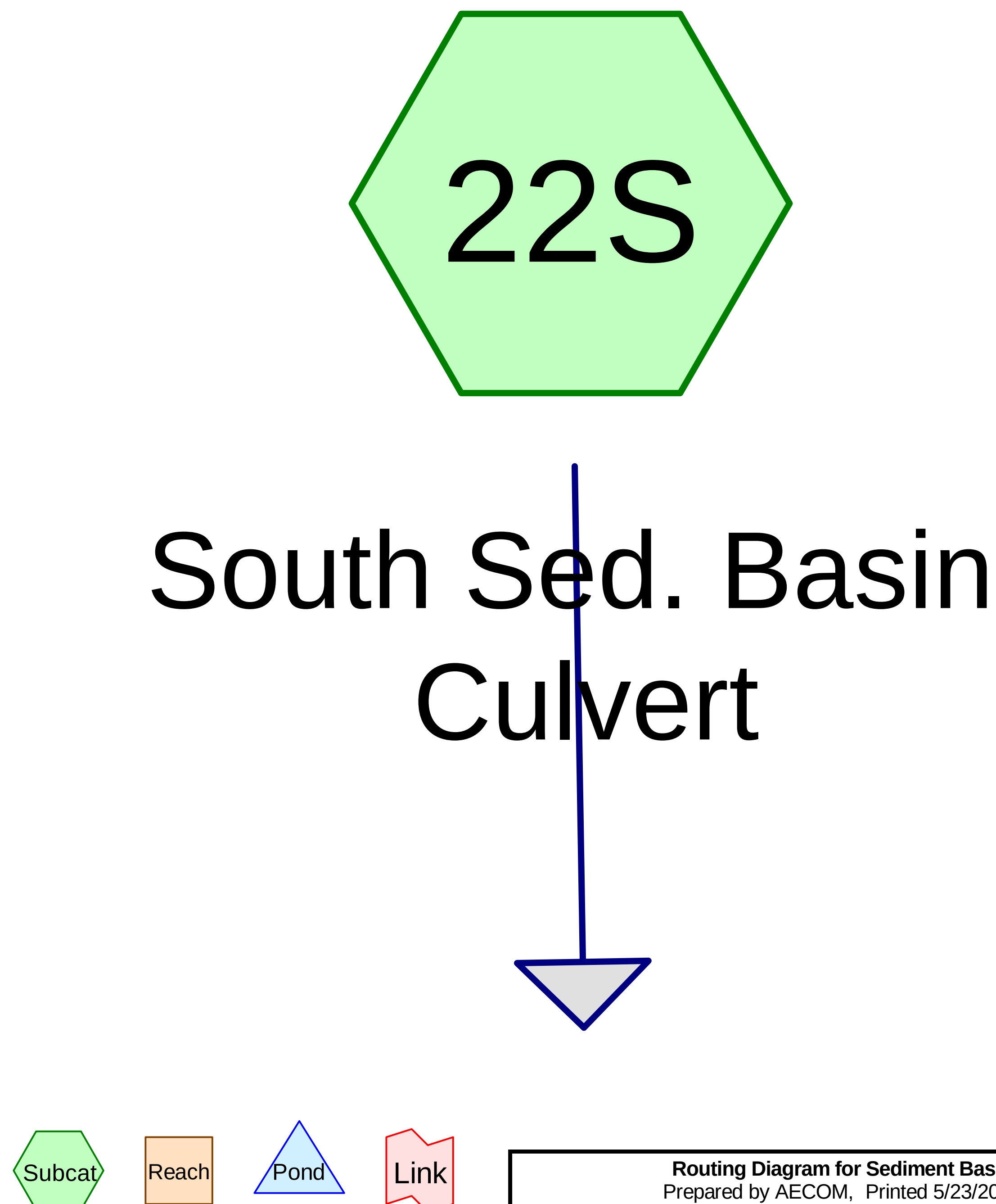
Aggregation Method: Dominant Condition

Component Percent Cutoff: *None Specified*

Tie-break Rule: Higher

ATTACHMENT C

South Sed. Basin Culvert



Routing Diagram for Sediment Basin
Prepared by AECOM, Printed 5/23/2017
HydroCAD® 10.00 s/n 01089 © 2013 HydroCAD Software Solutions LLC

Summary for Subcatchment 22S: South Sed. Basin Culvert

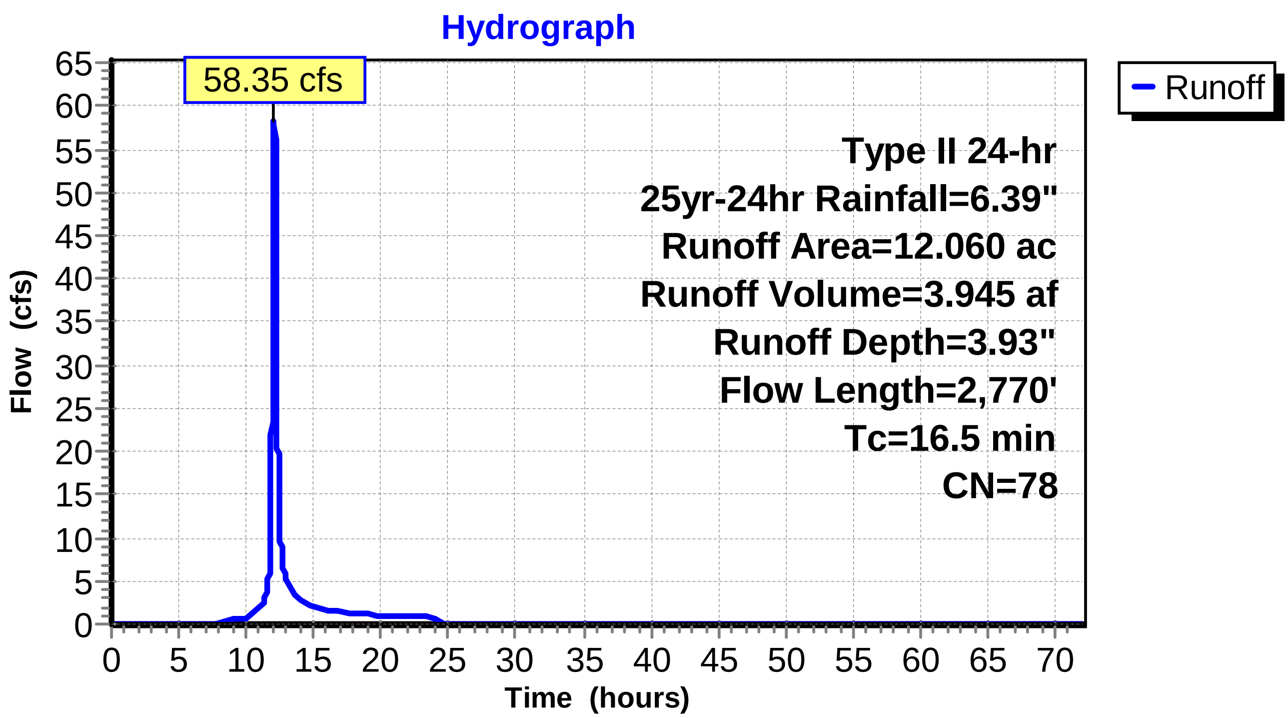
Runoff = 58.35 cfs @ 12.08 hrs, Volume= 3.945 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 22S: South Sed. Basin Culvert



Hydrograph for Subcatchment 22S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.04
8.00	0.77	0.01	0.18
9.00	0.94	0.04	0.45
10.00	1.16	0.10	0.82
11.00	1.50	0.23	1.94
12.00	4.24	2.08	**46.36**
13.00	4.93	2.65	**4.91**
14.00	5.24	2.92	2.80
15.00	5.45	3.10	2.13
16.00	5.62	3.25	1.68
17.00	5.76	3.37	1.44
18.00	5.89	3.48	1.27
19.00	5.99	3.57	1.11
20.00	6.08	3.65	0.94
21.00	6.16	3.72	0.87
22.00	6.24	3.79	0.84
23.00	6.32	3.86	0.81
24.00	**6.39**	**3.93**	0.77
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

HY-8 Culvert Analysis Report

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 58.35 cfs

Design Flow: 58.35 cfs

Maximum Flow: 58.35 cfs

Table 1 - Summary of Culvert Flows at Crossing: South Sed. Basin Culvert (improved

Headwater Elevation (ft)	Total Discharge (cfs)	2-30 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
464.29	58.35	58.35	0.00	1
465.75	84.31	84.31	0.00	Overtopping

Rating Curve Plot for Crossing: South Sed. Basin Culvert (improved inlet)

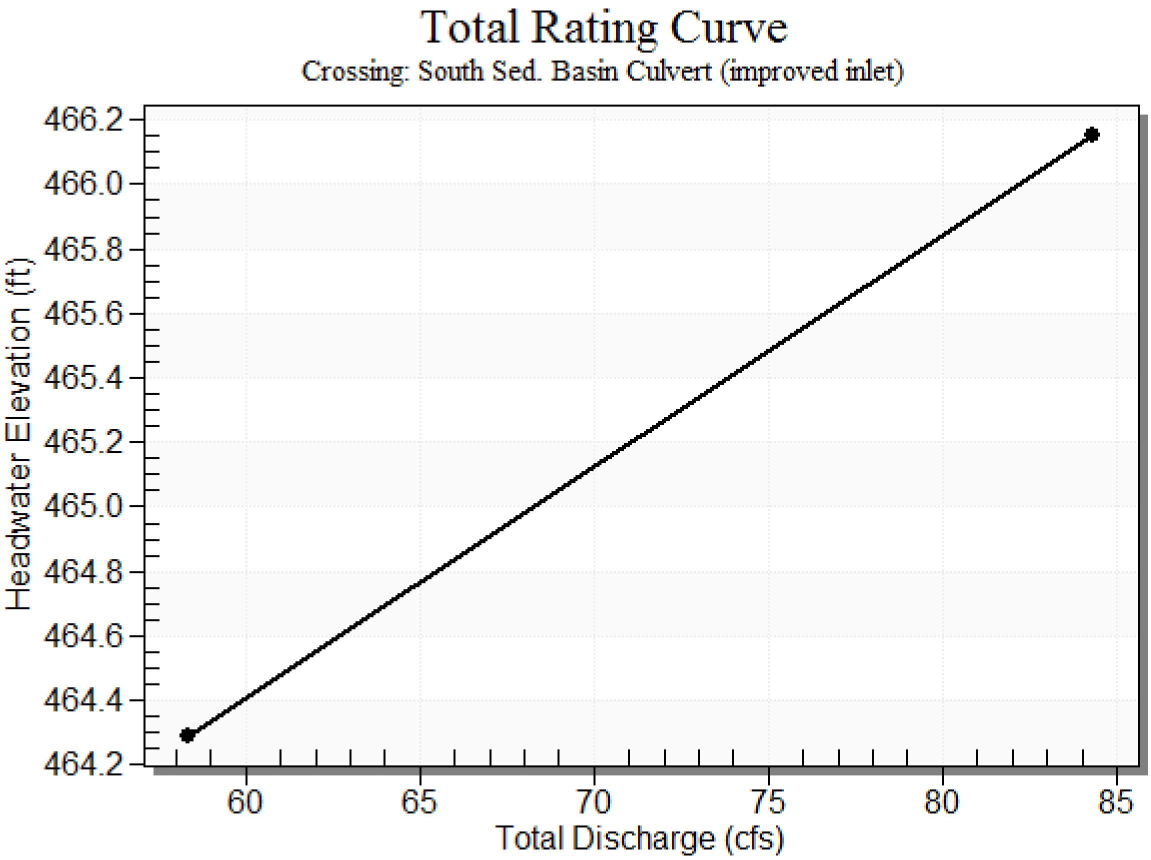


Table 2 - Culvert Summary Table: 2-30

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000
58.35	58.35	464.29	3.042	1.799	5-S2n	1.050	1.839	1.282	0.000	11.515	0.000

****

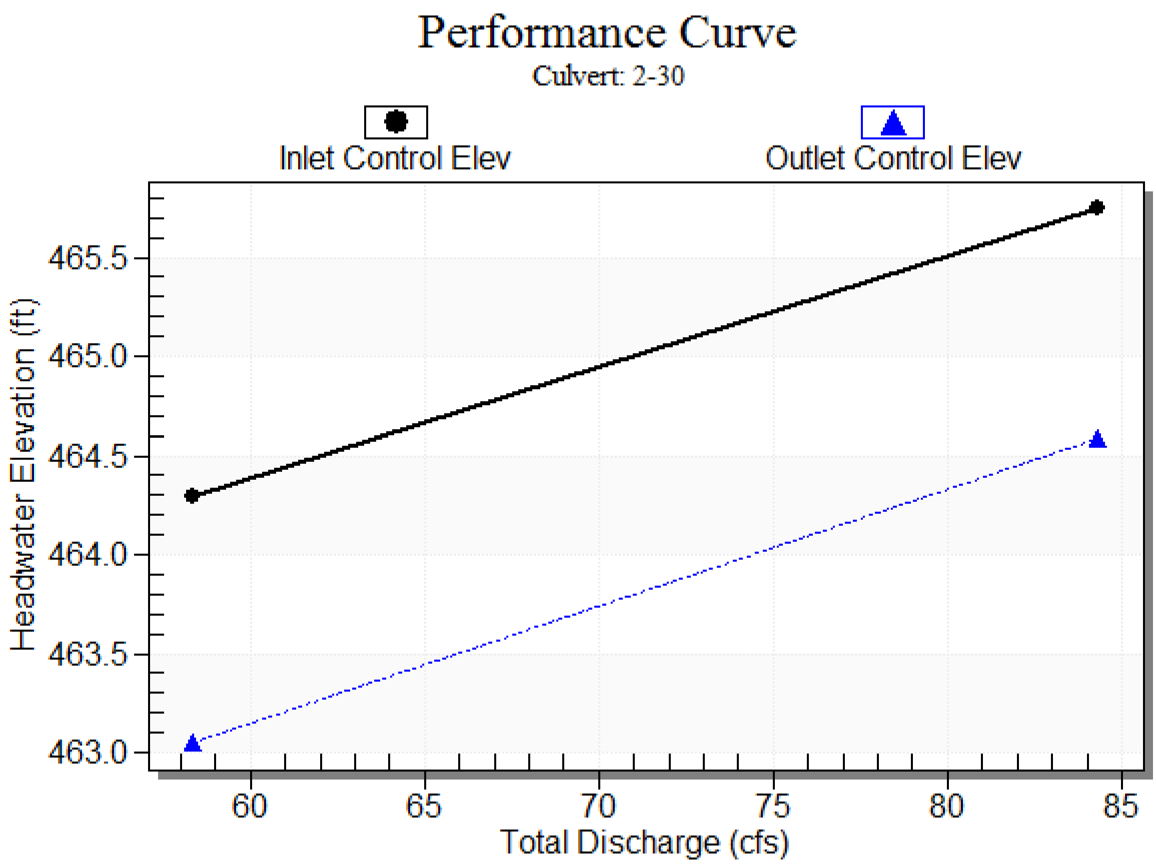
Straight Culvert

Inlet Elevation (invert): 461.25 ft, Outlet Elevation (invert): 460.25 ft

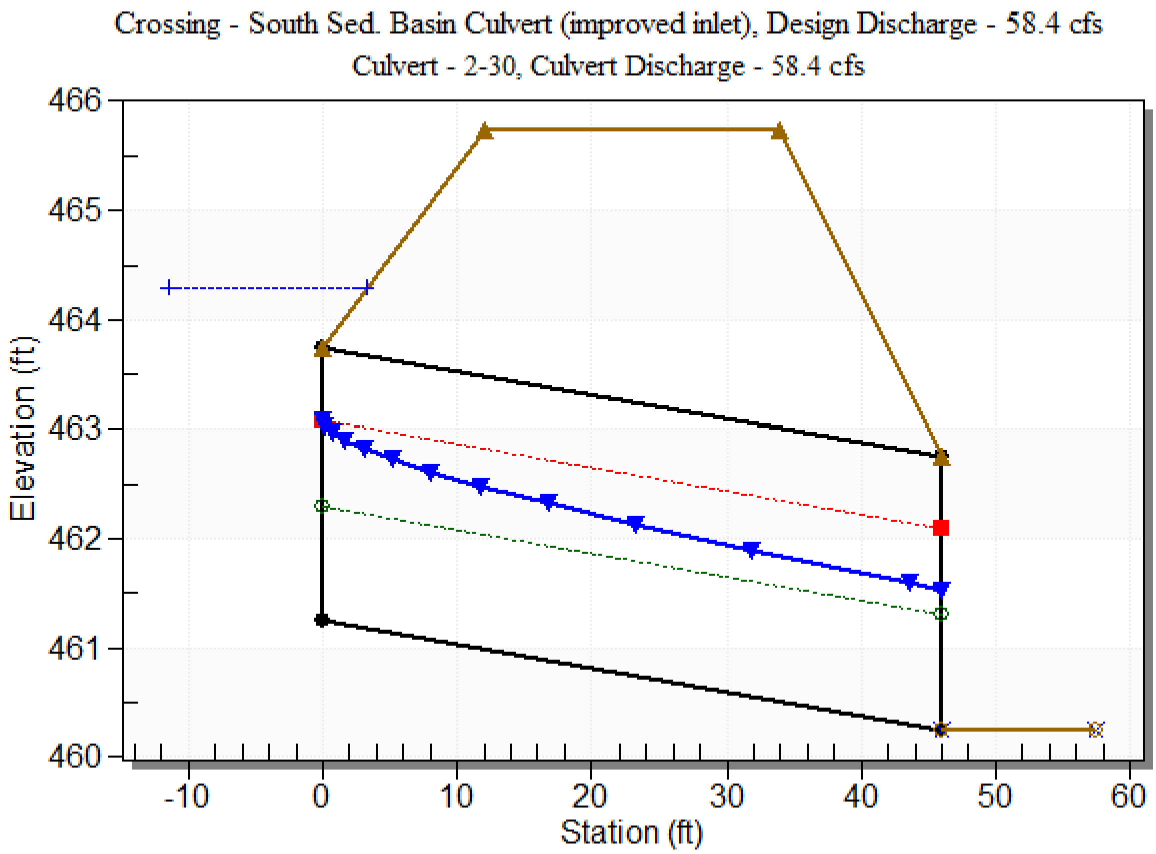
Culvert Length: 46.01 ft, Culvert Slope: 0.0217

****

Culvert Performance Curve Plot: 2-30



Water Surface Profile Plot for Culvert: 2-30



Site Data - 2-30

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 461.25 ft

Outlet Station: 46.00 ft

Outlet Elevation: 460.25 ft

Number of Barrels: 2

Culvert Data Summary - 2-30

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Corrugated PE

Embedment: 0.00 in

Barrel Manning's n: 0.0100

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: NONE

Table 3 - Downstream Channel Rating Curve (Crossing: South Sed. Basin Culvert (impro

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00
58.35	460.25	0.00

Tailwater Channel Data - South Sed. Basin Culvert (improved inlet)

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 460.25 ft

Roadway Data for Crossing: South Sed. Basin Culvert (improved inlet)

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 160.00 ft

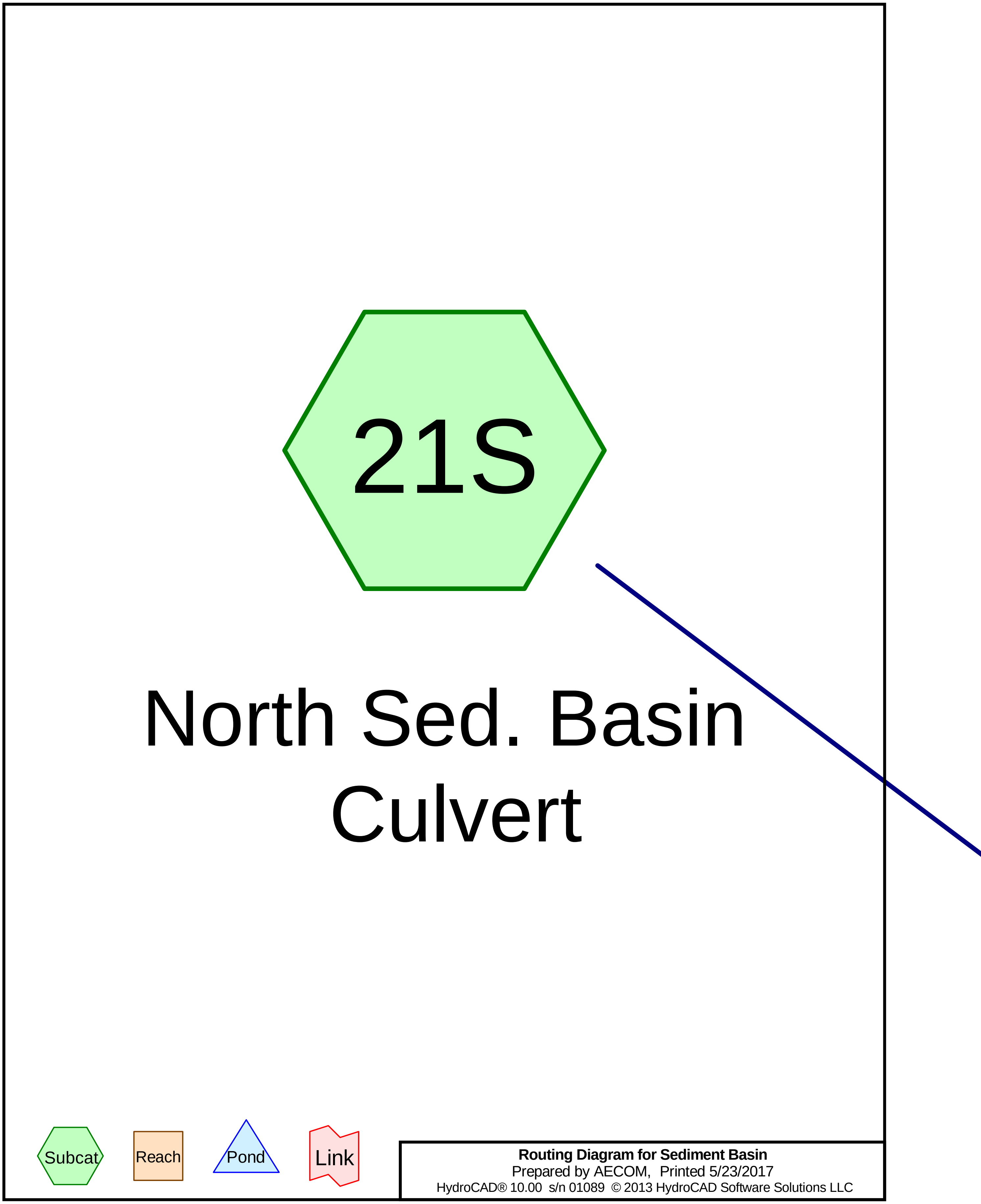
Crest Elevation: 465.75 ft

Roadway Surface: Gravel

Roadway Top Width: 22.00 ft

ATTACHMENT D

North Sed. Basin Culvert



Routing Diagram for Sediment Basin
Prepared by AECOM, Printed 5/23/2017
HydroCAD® 10.00 s/n 01089 © 2013 HydroCAD Software Solutions LLC

Summary for Subcatchment 21S: North Sed. Basin Culvert

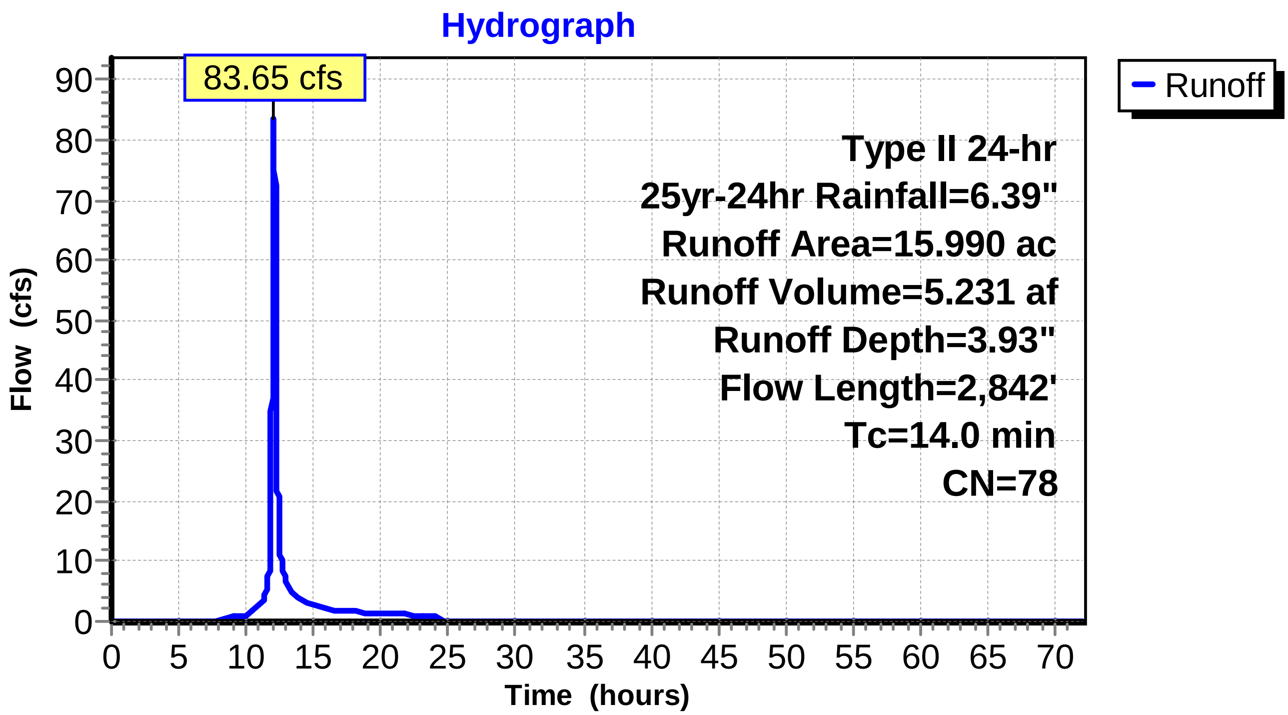
Runoff = 83.65 cfs @ 12.06 hrs, Volume= 5.231 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 21S: North Sed. Basin Culvert



Hydrograph for Subcatchment 21S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.06
8.00	0.77	0.01	0.25
9.00	0.94	0.04	0.62
10.00	1.16	0.10	1.12
11.00	1.50	0.23	2.67
12.00	4.24	2.08	**73.26**
13.00	4.93	2.65	**6.29**
14.00	5.24	2.92	3.64
15.00	5.45	3.10	2.81
16.00	5.62	3.25	2.20
17.00	5.76	3.37	1.90
18.00	5.89	3.48	1.68
19.00	5.99	3.57	1.46
20.00	6.08	3.65	1.23
21.00	6.16	3.72	1.15
22.00	6.24	3.79	1.11
23.00	6.32	3.86	1.07
24.00	**6.39**	**3.93**	1.02
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

HY-8 Culvert Analysis Report

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 83.65 cfs

Design Flow: 83.65 cfs

Maximum Flow: 83.65 cfs

Table 1 - Summary of Culvert Flows at Crossing: North Sed. Basin Culvert (improved

Headwater Elevation (ft)	Total Discharge (cfs)	3-30 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
466.67	83.65	83.65	0.00	1
468.25	126.46	126.46	0.00	Overtopping

Rating Curve Plot for Crossing: North Sed. Basin Culvert (improved inlet)

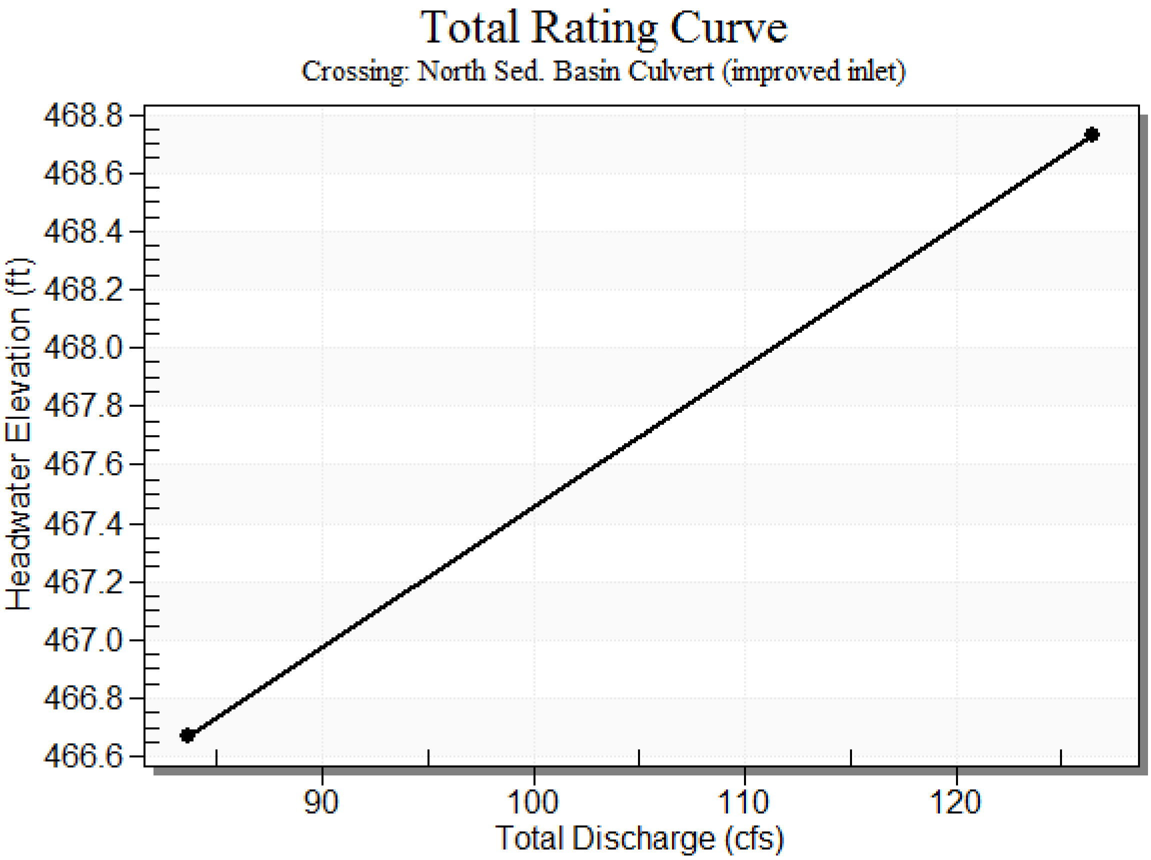


Table 2 - Culvert Summary Table: 3-30

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000
83.65	83.65	466.67	2.923	1.675	5-S2n	1.024	1.798	1.246	0.000	11.406	0.000

****

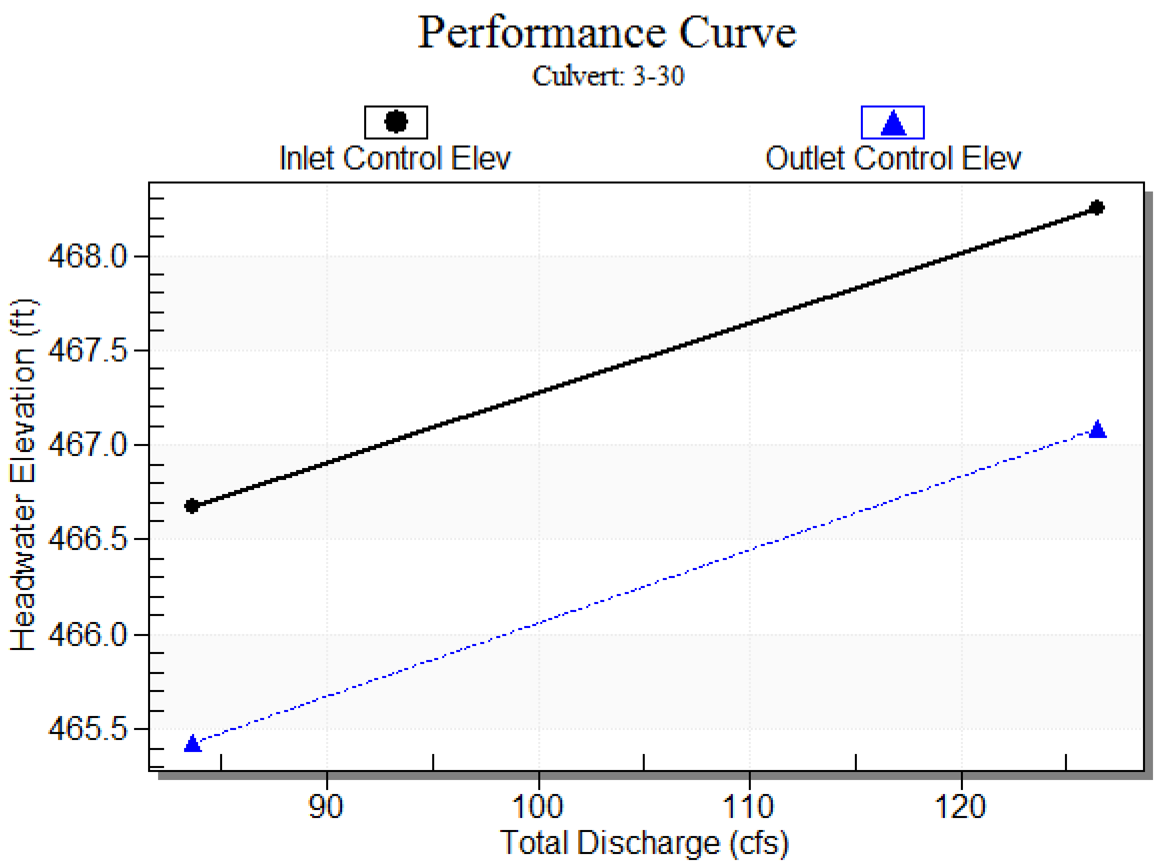
Straight Culvert

Inlet Elevation (invert): 463.75 ft,   Outlet Elevation (invert): 462.75 ft

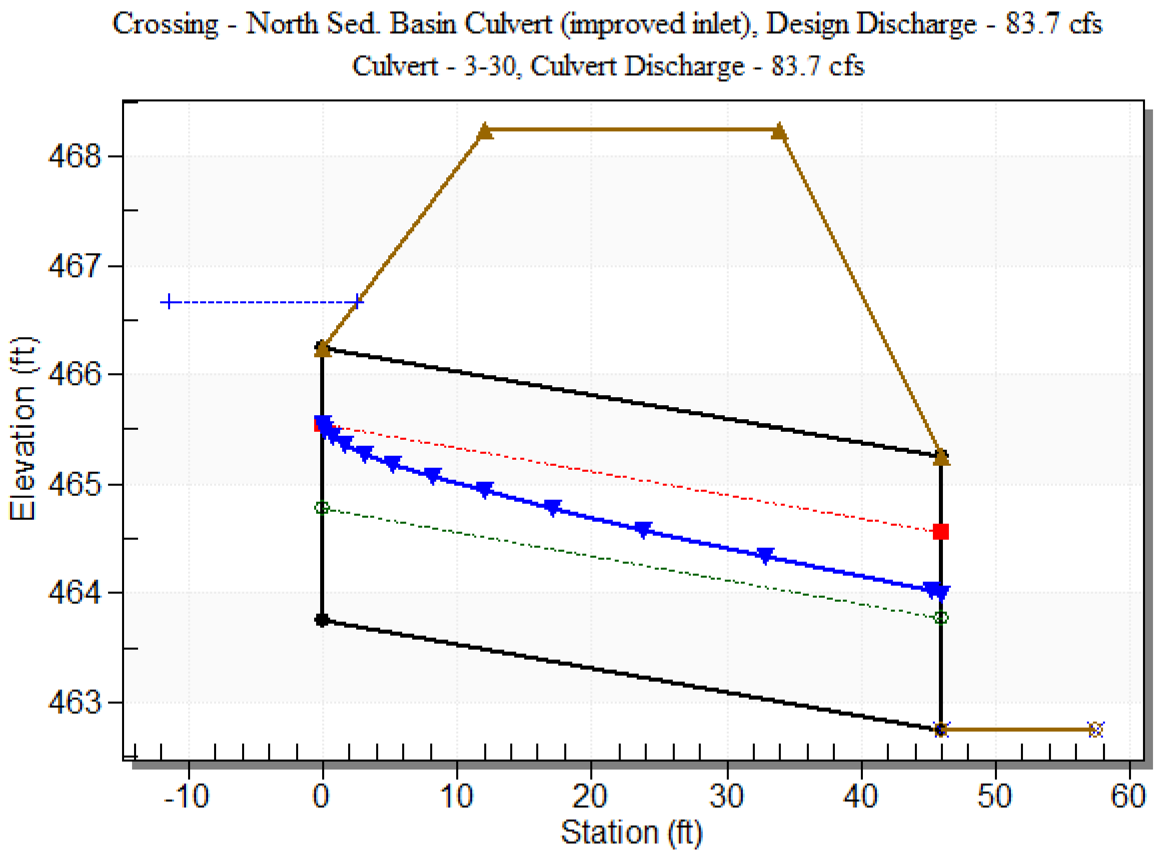
Culvert Length: 46.01 ft,   Culvert Slope: 0.0217

****

Culvert Performance Curve Plot: 3-30



Water Surface Profile Plot for Culvert: 3-30



Site Data - 3-30

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 463.75 ft

Outlet Station: 46.00 ft

Outlet Elevation: 462.75 ft

Number of Barrels: 3

Culvert Data Summary - 3-30

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Corrugated PE

Embedment: 0.00 in

Barrel Manning's n: 0.0100

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: NONE

Table 3 - Downstream Channel Rating Curve (Crossing: North Sed. Basin Culvert (impro

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00
83.65	462.75	0.00

Tailwater Channel Data - North Sed. Basin Culvert (improved inlet)

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 462.75 ft

Roadway Data for Crossing: North Sed. Basin Culvert (improved inlet)

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 160.00 ft

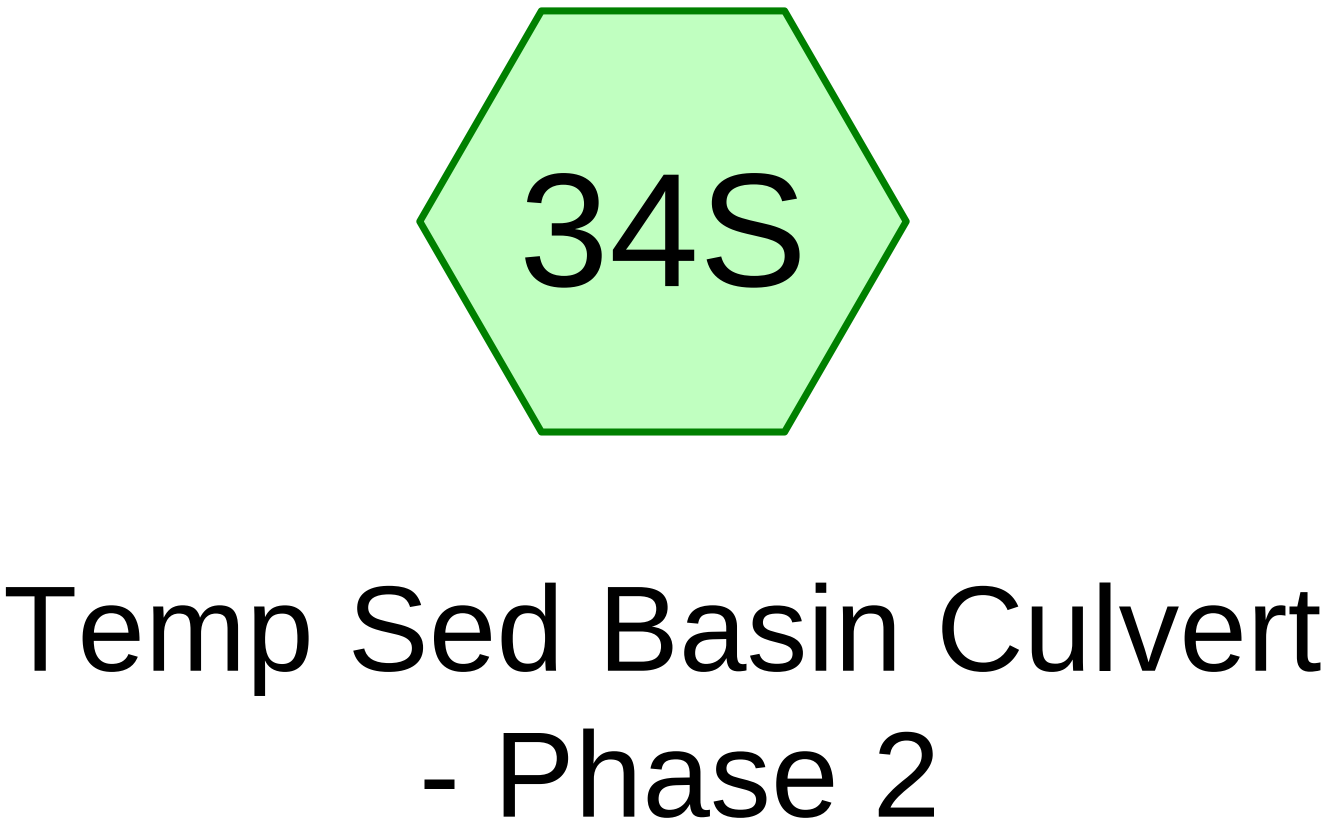
Crest Elevation: 468.25 ft

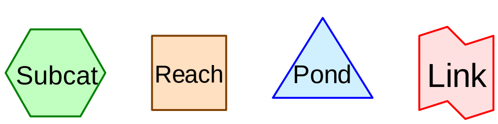
Roadway Surface: Gravel

Roadway Top Width: 22.00 ft

ATTACHMENT E

Temporary Sed. Basin Culvert





Routing Diagram for Temp Culvert
Prepared by AECOM, Printed 5/23/2017

Summary for Subcatchment 34S: Temp Sed Basin Culvert - Phase 2

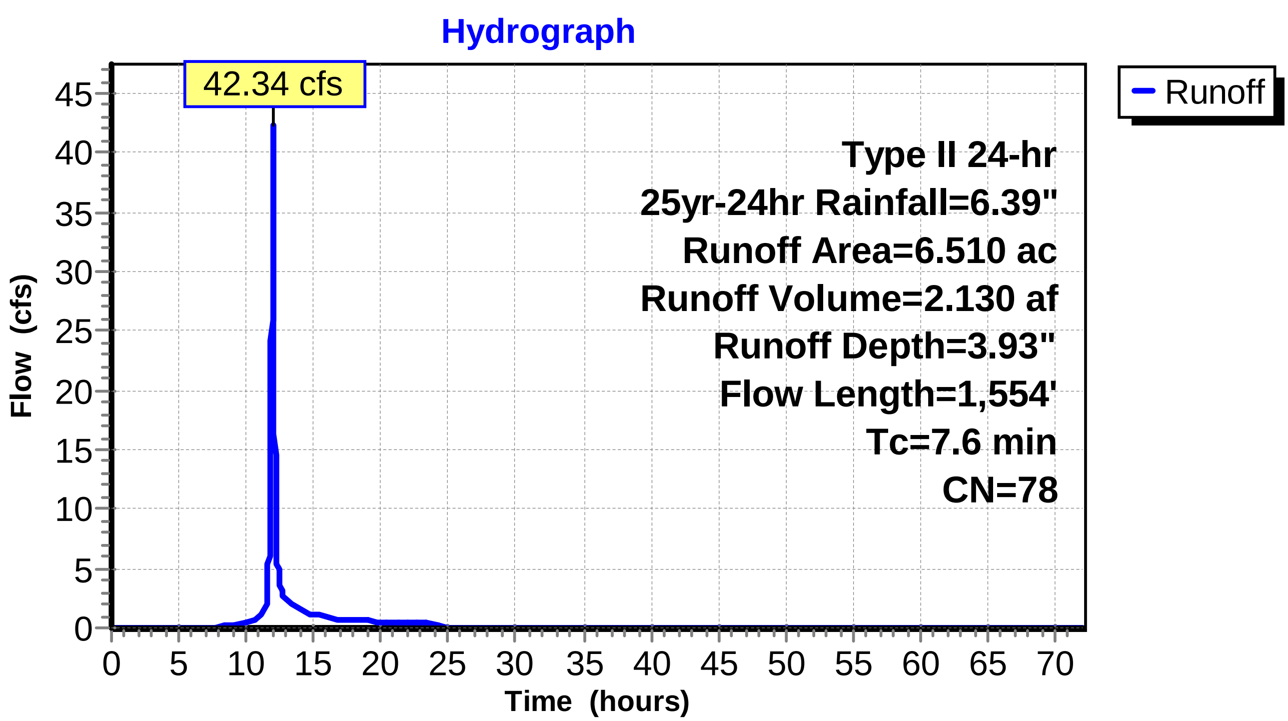
Runoff = 42.34 cfs @ 11.99 hrs, Volume= 2.130 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	6.510	78	
	6.510		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
4.3	100	0.3333	0.39		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.8	182	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
1.1	645	0.0300	10.21	367.62	**Channel Flow, Diversion Channel 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.4	627	0.0154	7.32	263.39	**Channel Flow, Diversion Channel 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
7.6	1,554	Total			

Subcatchment 34S: Temp Sed Basin Culvert - Phase 2



Hydrograph for Subcatchment 34S: Temp Sed Basin Culvert - Phase 2

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.03
8.00	0.77	0.01	0.11
9.00	0.94	0.04	0.27
10.00	1.16	0.10	0.49
11.00	1.50	0.23	**1.19**
12.00	4.24	2.08	**42.03**
13.00	4.93	2.65	2.38
14.00	5.24	2.92	1.41
15.00	5.45	3.10	1.12
16.00	5.62	3.25	0.87
17.00	5.76	3.37	0.76
18.00	5.89	3.48	0.67
19.00	5.99	3.57	0.58
20.00	6.08	3.65	0.49
21.00	6.16	3.72	0.47
22.00	6.24	3.79	0.45
23.00	6.32	3.86	0.43
24.00	**6.39**	**3.93**	0.41
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

HY-8 Culvert Analysis Report

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 42.34 cfs

Design Flow: 42.34 cfs

Maximum Flow: 42.34 cfs

Table 1 - Summary of Culvert Flows at Crossing: Temp Culvert

Headwater Elevation (ft)	Total Discharge (cfs)	2-30 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
462.39	42.34	42.34	0.00	1
463.00	56.97	56.97	0.00	Overtopping

Rating Curve Plot for Crossing: Temp Culvert

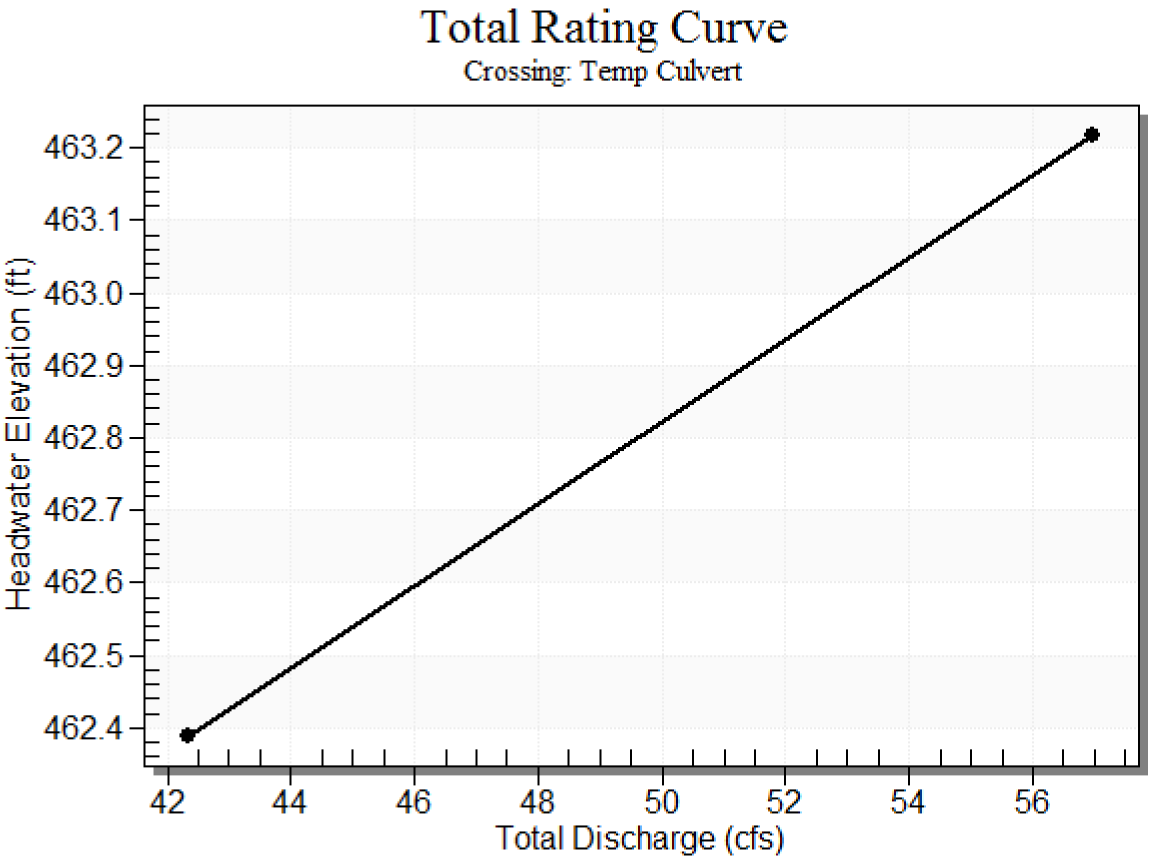


Table 2 - Culvert Summary Table: 2-30

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000
42.34	42.34	462.39	2.390	0.211	1-S2n	0.962	1.561	1.038	0.000	10.617	0.000

****

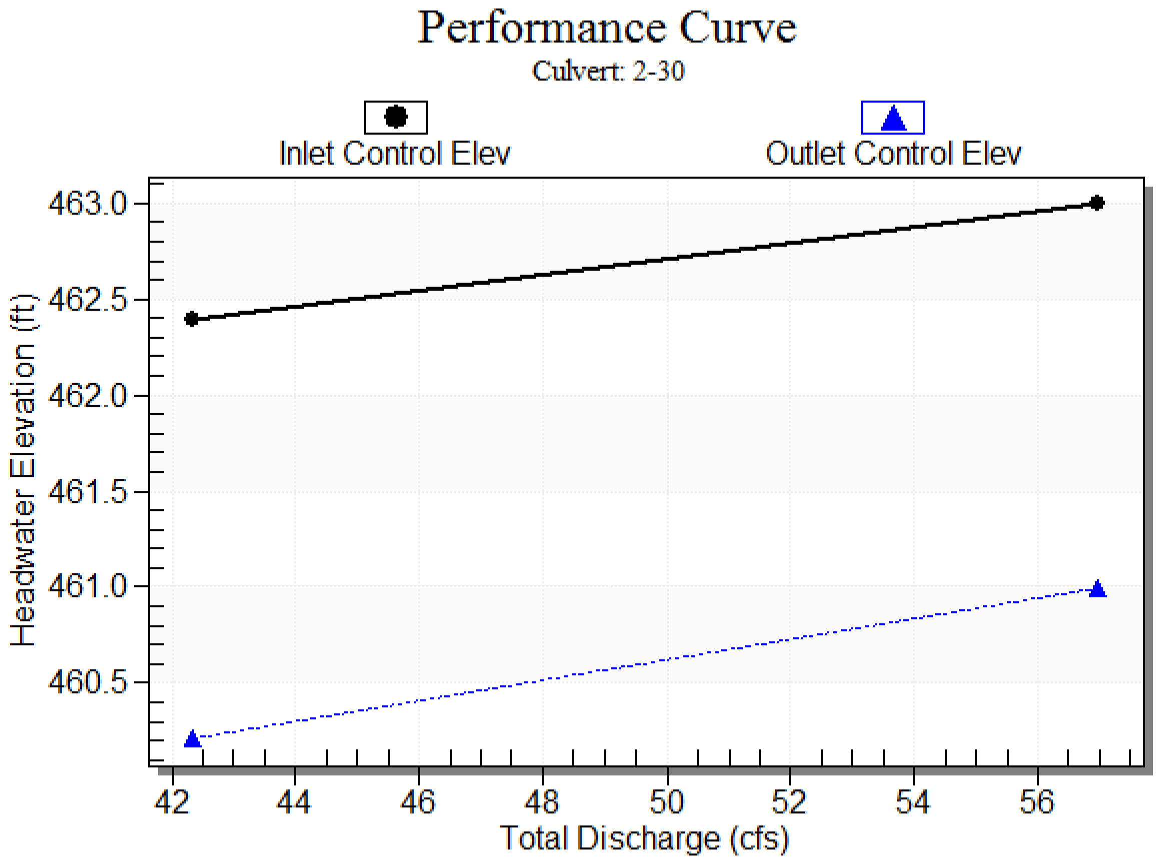
Straight Culvert

Inlet Elevation (invert): 460.00 ft,   Outlet Elevation (invert): 458.00 ft

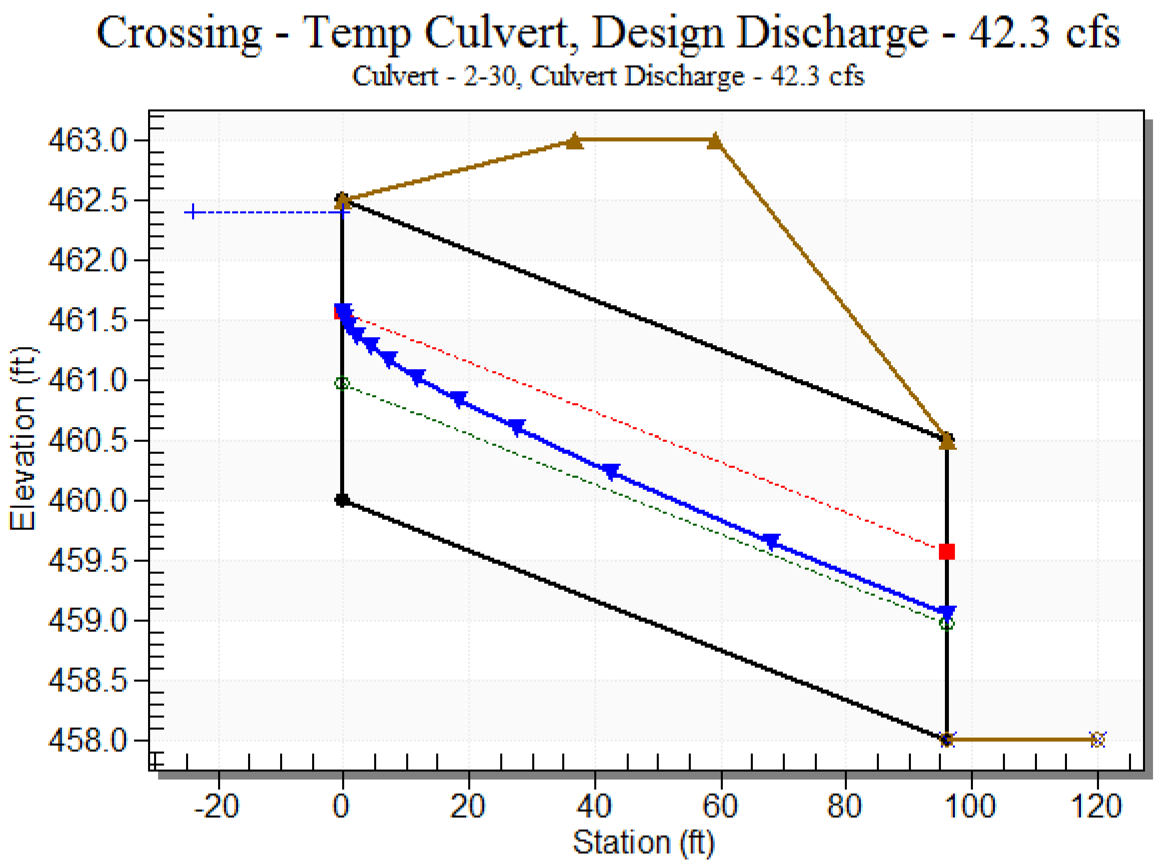
Culvert Length: 96.02 ft,   Culvert Slope: 0.0208

****

Culvert Performance Curve Plot: 2-30



Water Surface Profile Plot for Culvert: 2-30



Site Data - 2-30

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 460.00 ft

Outlet Station: 96.00 ft

Outlet Elevation: 458.00 ft

Number of Barrels: 2

Culvert Data Summary - 2-30

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Table 3 - Downstream Channel Rating Curve (Crossing: Temp Culvert)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00
42.34	458.00	0.00

Tailwater Channel Data - Temp Culvert

Tailwater Channel Option: Enter Constant Tailwater Elevation

Constant Tailwater Elevation: 458.00 ft

Roadway Data for Crossing: Temp Culvert

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 160.00 ft

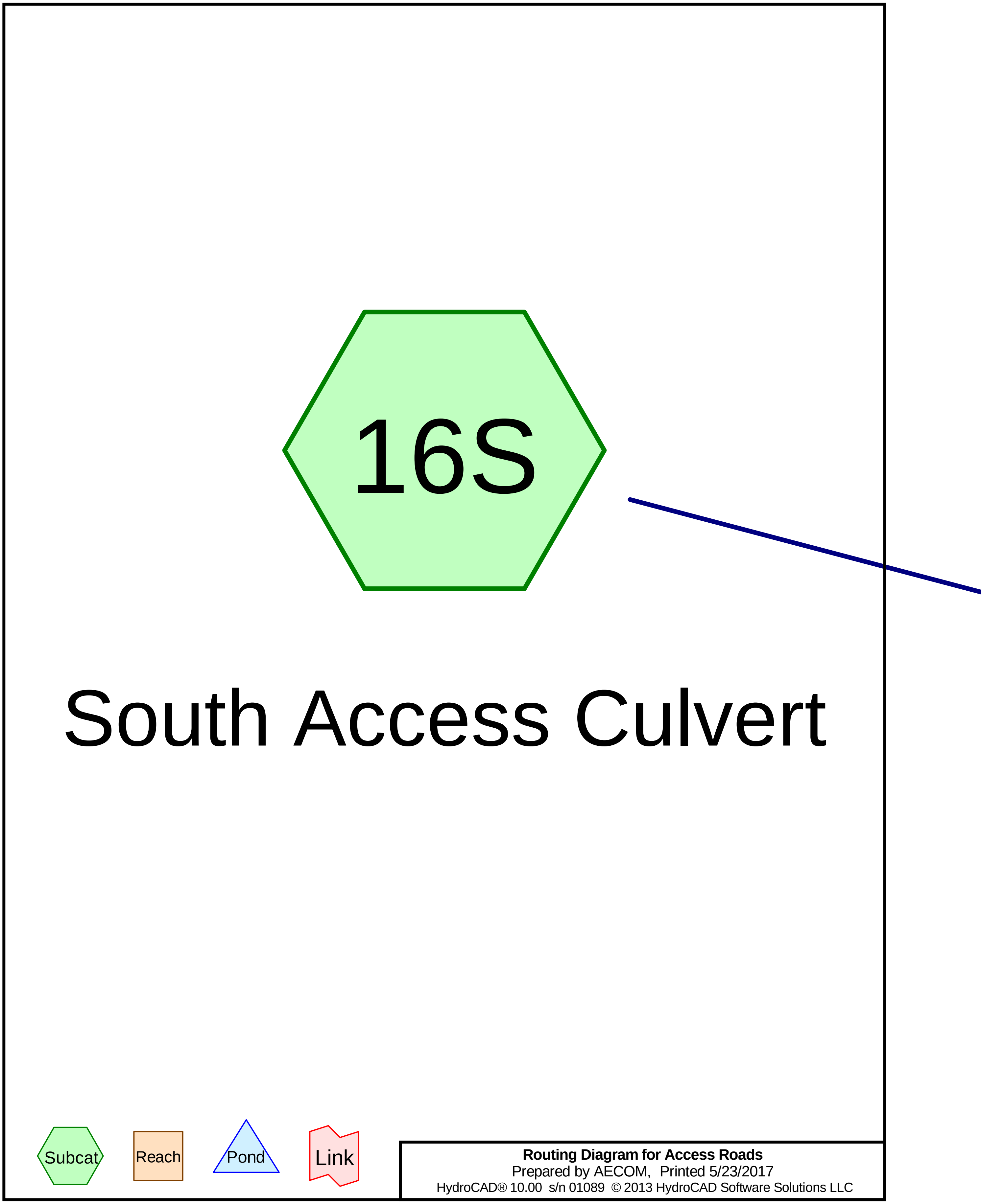
Crest Elevation: 463.00 ft

Roadway Surface: Gravel

Roadway Top Width: 22.00 ft

ATTACHMENT F

South Access Road Culvert


16S
South Access Culvert
Subcat
Reach
Pond
Link
Routing Diagram for Access Roads
Prepared by AECOM, Printed 5/23/2017
HydroCAD® 10.00 s/n 01089 © 2013 HydroCAD Software Solutions LLC

Summary for Subcatchment 16S: South Access Culvert

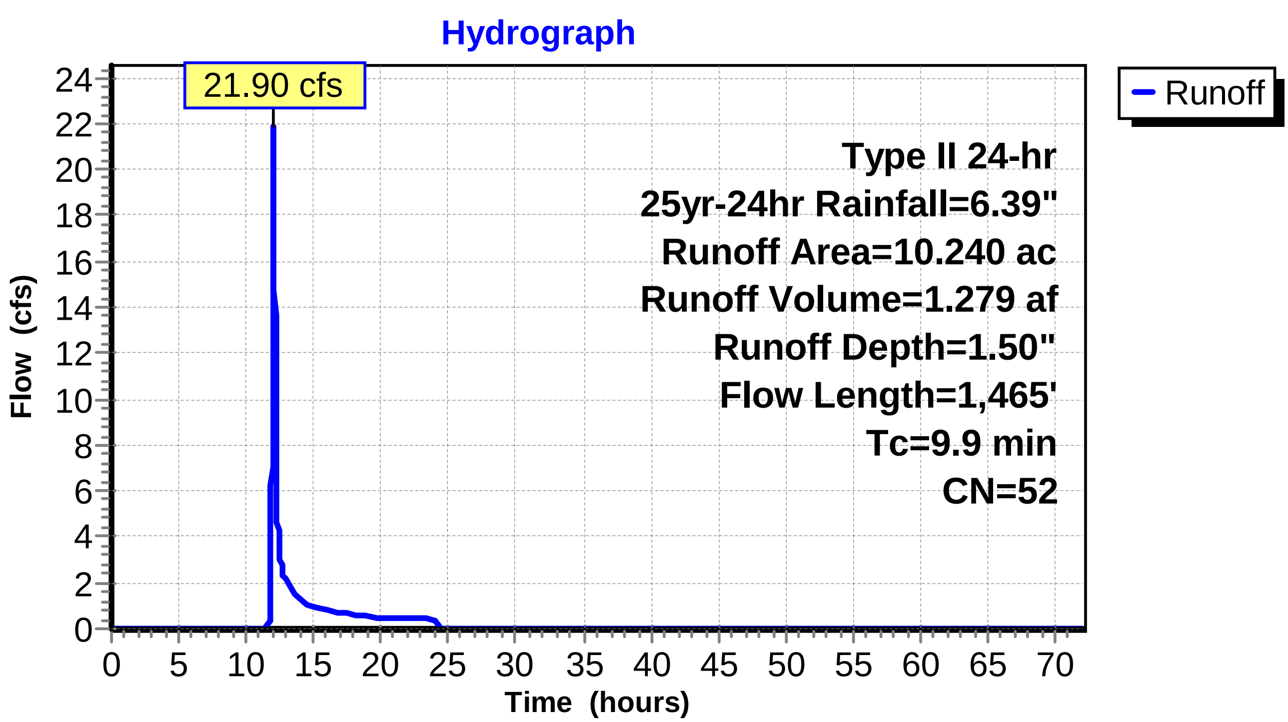
Runoff = 21.90 cfs @ 12.03 hrs, Volume= 1.279 af, Depth= 1.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

Area (ac)	CN	Description
9.240	49	50-75% Grass cover, Fair, HSG A
1.000	77	Newly graded area, HSG A
10.240	52	Weighted Average
10.240		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
7.6	100	0.0800	0.22		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.6	177	0.0800	4.55		**Shallow Concentrated Flow, Shallow Concentrated Flow** Unpaved Kv= 16.1 fps
0.2	235	0.2490	21.19	211.86	**Channel Flow, Valley 1** Area= 10.0 sf Perim= 10.0' r= 1.00' n= 0.035
1.5	953	0.0661	10.92	109.16	**Channel Flow, Valley 2** Area= 10.0 sf Perim= 10.0' r= 1.00' n= 0.035
9.9	1,465	Total			

Subcatchment 16S: South Access Culvert



Hydrograph for Subcatchment 16S: South Access Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.00
8.00	0.77	0.00	0.00
9.00	0.94	0.00	0.00
10.00	1.16	0.00	0.00
11.00	1.50	0.00	0.00
12.00	4.24	0.49	**20.53**
13.00	4.93	0.77	**1.97**
14.00	5.24	0.91	1.22
15.00	5.45	1.01	0.99
16.00	5.62	1.10	0.79
17.00	5.76	1.17	0.70
18.00	5.89	1.23	0.62
19.00	5.99	1.29	0.55
20.00	6.08	1.33	0.47
21.00	6.16	1.38	0.44
22.00	6.24	1.42	0.43
23.00	6.32	1.46	0.42
24.00	**6.39**	**1.50**	0.40
25.00	6.39	1.50	0.00
26.00	6.39	1.50	0.00
27.00	6.39	1.50	0.00
28.00	6.39	1.50	0.00
29.00	6.39	1.50	0.00
30.00	6.39	1.50	0.00
31.00	6.39	1.50	0.00
32.00	6.39	1.50	0.00
33.00	6.39	1.50	0.00
34.00	6.39	1.50	0.00
35.00	6.39	1.50	0.00
36.00	6.39	1.50	0.00
37.00	6.39	1.50	0.00
38.00	6.39	1.50	0.00
39.00	6.39	1.50	0.00
40.00	6.39	1.50	0.00
41.00	6.39	1.50	0.00
42.00	6.39	1.50	0.00
43.00	6.39	1.50	0.00
44.00	6.39	1.50	0.00
45.00	6.39	1.50	0.00
46.00	6.39	1.50	0.00
47.00	6.39	1.50	0.00
48.00	6.39	1.50	0.00
49.00	6.39	1.50	0.00
50.00	6.39	1.50	0.00
51.00	6.39	1.50	0.00
52.00	6.39	1.50	0.00
53.00	6.39	1.50	0.00
54.00	6.39	1.50	0.00
55.00	6.39	1.50	0.00
56.00	6.39	1.50	0.00
57.00	6.39	1.50	0.00
58.00	6.39	1.50	0.00
59.00	6.39	1.50	0.00
60.00	6.39	1.50	0.00
61.00	6.39	1.50	0.00
62.00	6.39	1.50	0.00
63.00	6.39	1.50	0.00
64.00	6.39	1.50	0.00
65.00	6.39	1.50	0.00
66.00	6.39	1.50	0.00
67.00	6.39	1.50	0.00
68.00	6.39	1.50	0.00
69.00	6.39	1.50	0.00
70.00	6.39	1.50	0.00
71.00	6.39	1.50	0.00
72.00	6.39	1.50	0.00

HY-8 Culvert Analysis Report

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 21.9 cfs

Design Flow: 21.9 cfs

Maximum Flow: 21.9 cfs

Table 1 - Summary of Culvert Flows at Crossing: South Access Road

Headwater Elevation (ft)	Total Discharge (cfs)	1-18in RCP Discharge (cfs)	Roadway Discharge (cfs)	Iterations
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
475.24	21.90	21.90	0.00	1
507.53	40.05	40.05	0.00	Overtopping

Rating Curve Plot for Crossing: South Access Road

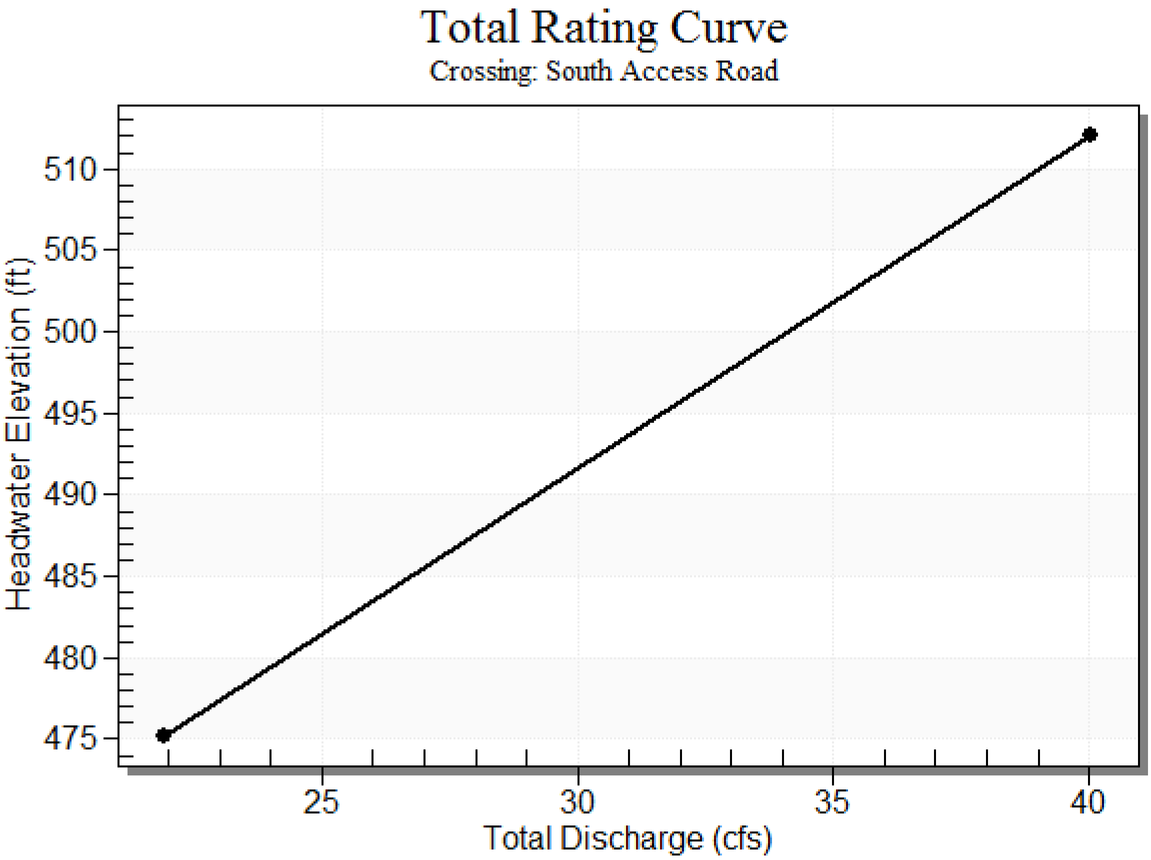


Table 2 - Culvert Summary Table: 1-18in RCP

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155
21.90	21.90	475.24	7.229	3.751	5-S2n	1.135	1.500	1.135	1.190	14.863	5.155

****

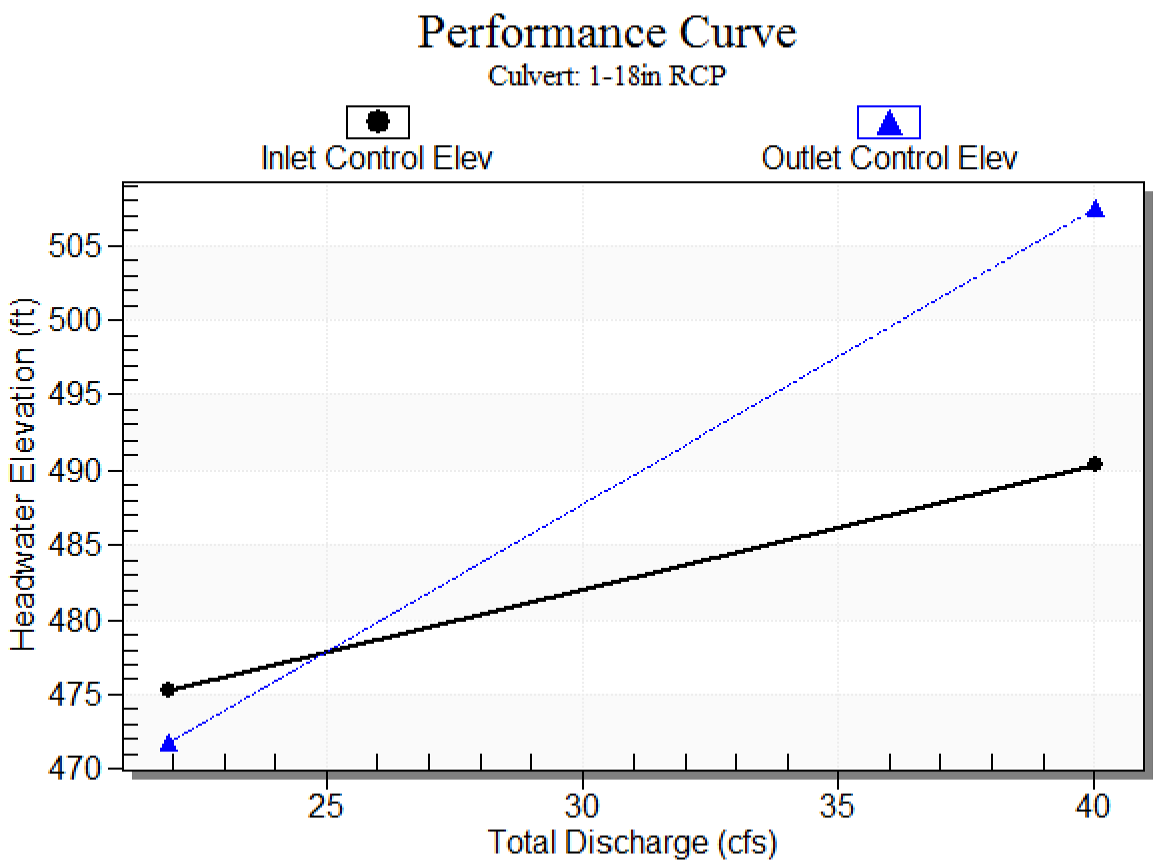
Straight Culvert

Inlet Elevation (invert): 468.01 ft,   Outlet Elevation (invert): 455.00 ft

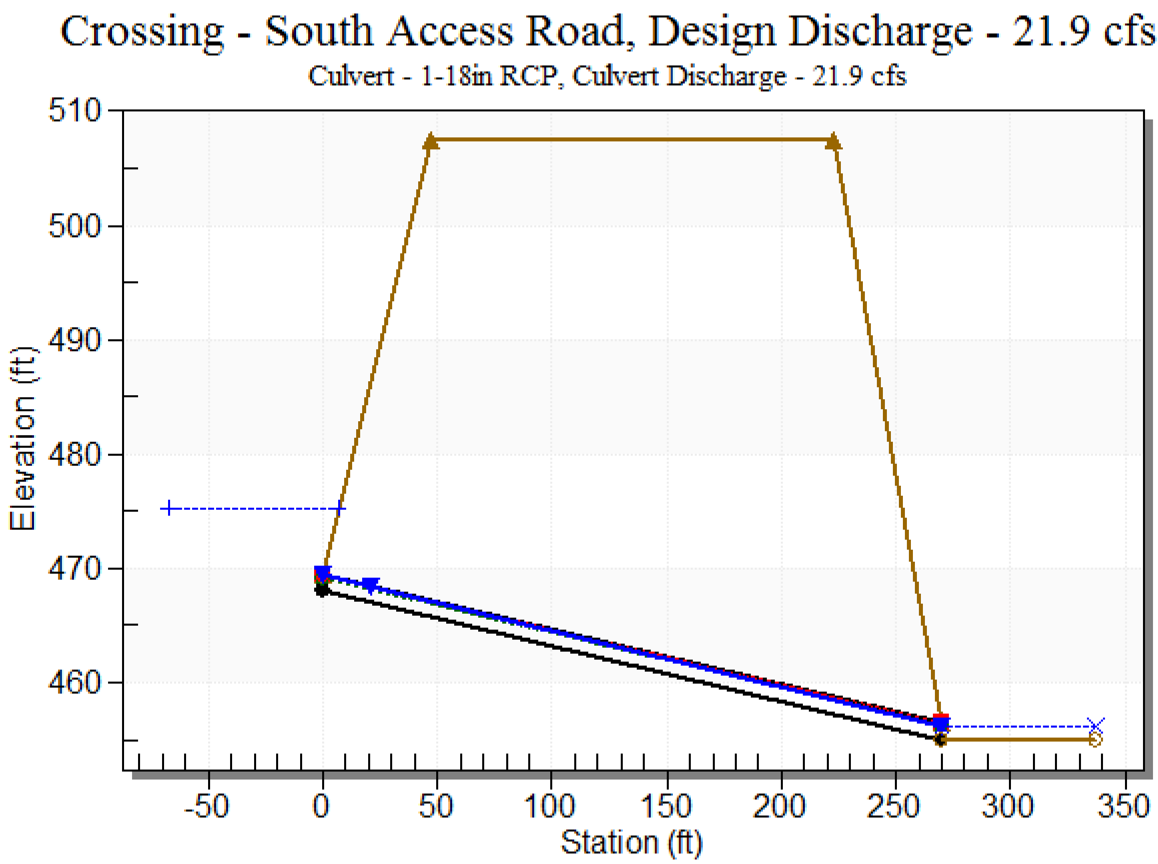
Culvert Length: 270.31 ft,   Culvert Slope: 0.0482

****

Culvert Performance Curve Plot: 1-18in RCP



Water Surface Profile Plot for Culvert: 1-18in RCP



Site Data - 1-18in RCP

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 468.01 ft

Outlet Station: 270.00 ft

Outlet Elevation: 455.00 ft

Number of Barrels: 1

Culvert Data Summary - 1-18in RCP

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0130

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Table 3 - Downstream Channel Rating Curve (Crossing: South Access Road)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)	Velocity (ft/s)	Shear (psf)	Froude Number
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18
21.90	456.19	1.19	5.15	2.35	1.18

Tailwater Channel Data - South Access Road

Tailwater Channel Option: Triangular Channel

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0316

Channel Manning's n: 0.0350

Channel Invert Elevation: 455.00 ft

Roadway Data for Crossing: South Access Road

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 22.00 ft

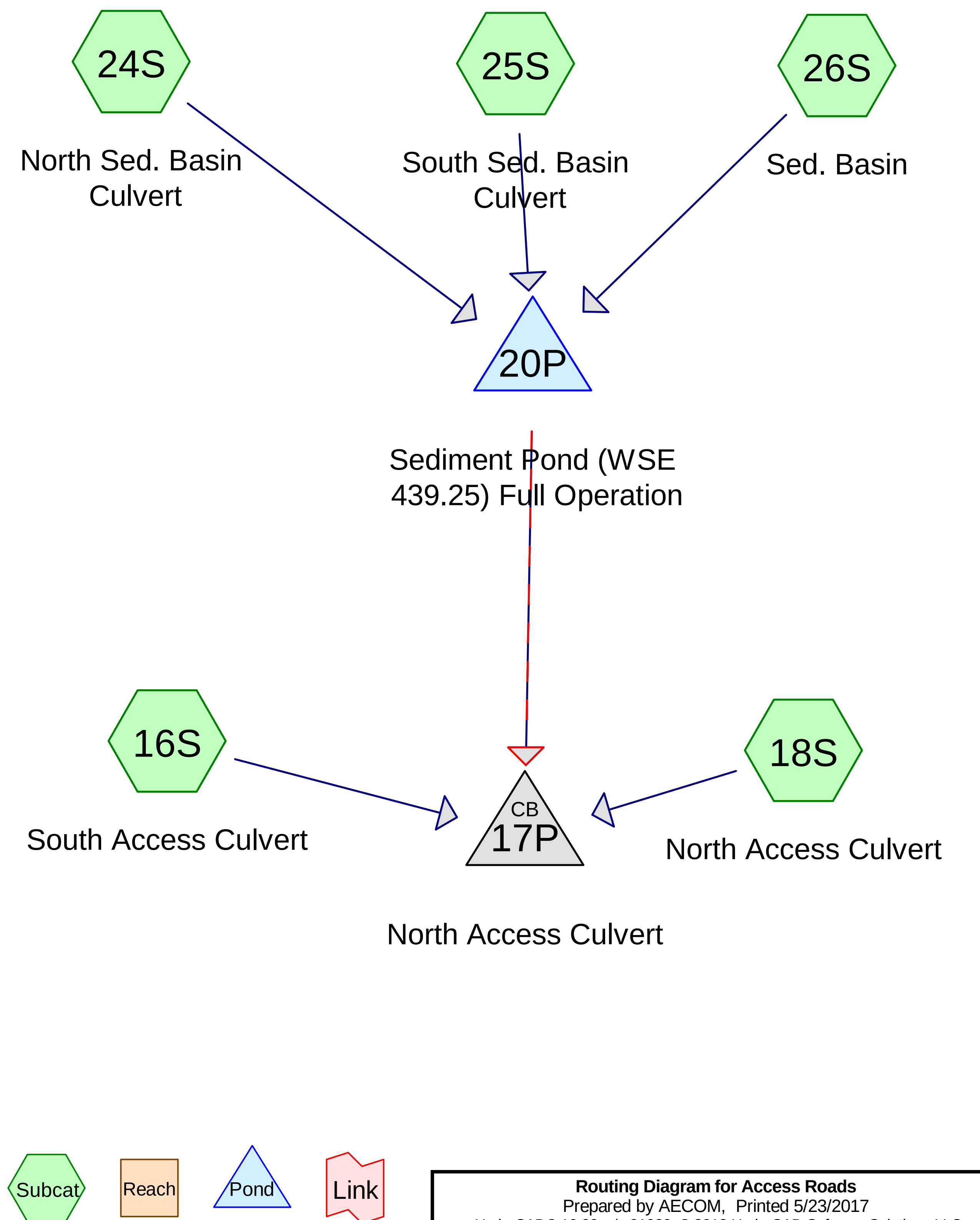
Crest Elevation: 507.53 ft

Roadway Surface: Gravel

Roadway Top Width: 175.00 ft

ATTACHMENT G

North Access Road Culvert



Routing Diagram for Access Roads
Prepared by AECOM, Printed 5/23/2017
HydroCAD® 10.00 s/n 01089 © 2013 HydroCAD Software Solutions LLC

Area Listing (all nodes)

Area (acres)	CN	Description (subcatchment-numbers)
29.550	78	(24S, 25S, 26S)
1.500	98	(26S)
16.500	49	50-75% Grass cover, Fair, HSG A (16S, 18S)
1.000	77	Newly graded area, HSG A (16S)
48.550	**69**	**TOTAL AREA**

Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
17.500	HSG A	16S, 18S
0.000	HSG B	
0.000	HSG C	
0.000	HSG D	
31.050	Other	24S, 25S, 26S
48.550		**TOTAL AREA**

Summary for Subcatchment 16S: South Access Culvert

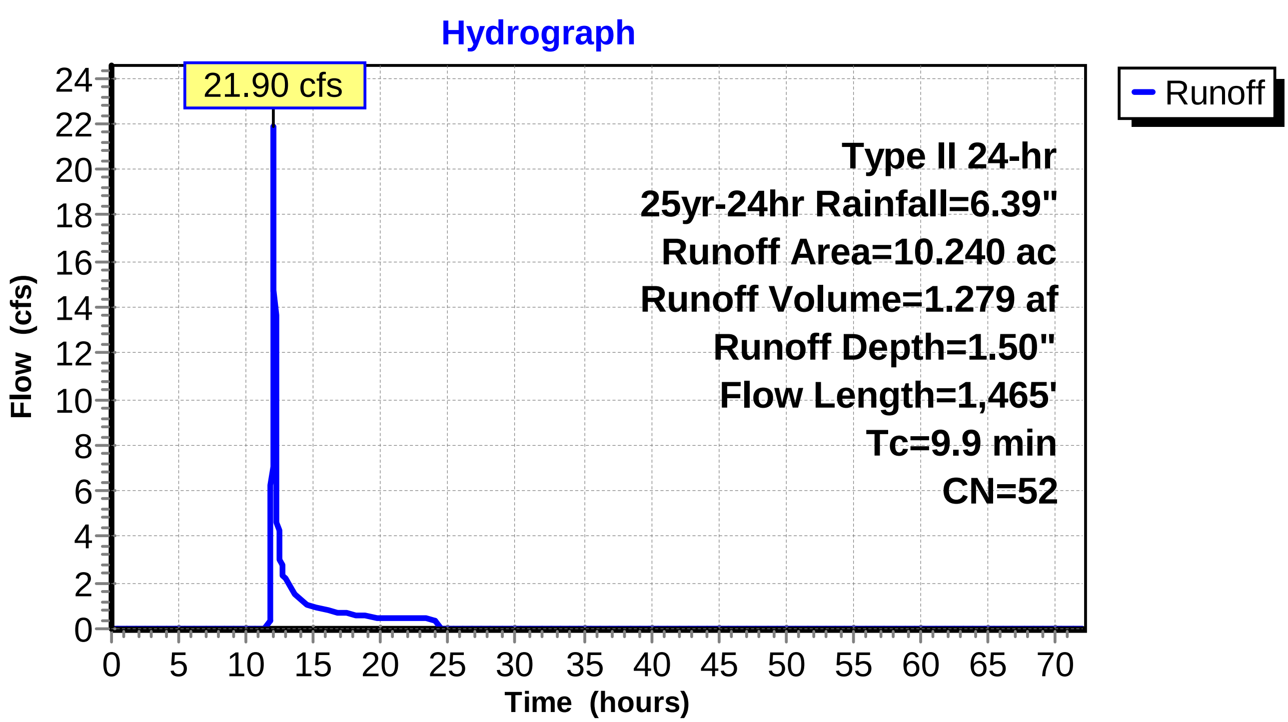
Runoff = 21.90 cfs @ 12.03 hrs, Volume= 1.279 af, Depth= 1.50"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

Area (ac)	CN	Description
9.240	49	50-75% Grass cover, Fair, HSG A
1.000	77	Newly graded area, HSG A
10.240	52	Weighted Average
10.240		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
7.6	100	0.0800	0.22		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.6	177	0.0800	4.55		**Shallow Concentrated Flow, Shallow Concentrated Flow** Unpaved Kv= 16.1 fps
0.2	235	0.2490	21.19	211.86	**Channel Flow, Valley 1** Area= 10.0 sf Perim= 10.0' r= 1.00' n= 0.035
1.5	953	0.0661	10.92	109.16	**Channel Flow, Valley 2** Area= 10.0 sf Perim= 10.0' r= 1.00' n= 0.035
9.9	1,465	Total			

Subcatchment 16S: South Access Culvert



Hydrograph for Subcatchment 16S: South Access Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.00
8.00	0.77	0.00	0.00
9.00	0.94	0.00	0.00
10.00	1.16	0.00	0.00
11.00	1.50	0.00	0.00
12.00	4.24	0.49	**20.53**
13.00	4.93	0.77	**1.97**
14.00	5.24	0.91	1.22
15.00	5.45	1.01	0.99
16.00	5.62	1.10	0.79
17.00	5.76	1.17	0.70
18.00	5.89	1.23	0.62
19.00	5.99	1.29	0.55
20.00	6.08	1.33	0.47
21.00	6.16	1.38	0.44
22.00	6.24	1.42	0.43
23.00	6.32	1.46	0.42
24.00	**6.39**	**1.50**	0.40
25.00	6.39	1.50	0.00
26.00	6.39	1.50	0.00
27.00	6.39	1.50	0.00
28.00	6.39	1.50	0.00
29.00	6.39	1.50	0.00
30.00	6.39	1.50	0.00
31.00	6.39	1.50	0.00
32.00	6.39	1.50	0.00
33.00	6.39	1.50	0.00
34.00	6.39	1.50	0.00
35.00	6.39	1.50	0.00
36.00	6.39	1.50	0.00
37.00	6.39	1.50	0.00
38.00	6.39	1.50	0.00
39.00	6.39	1.50	0.00
40.00	6.39	1.50	0.00
41.00	6.39	1.50	0.00
42.00	6.39	1.50	0.00
43.00	6.39	1.50	0.00
44.00	6.39	1.50	0.00
45.00	6.39	1.50	0.00
46.00	6.39	1.50	0.00
47.00	6.39	1.50	0.00
48.00	6.39	1.50	0.00
49.00	6.39	1.50	0.00
50.00	6.39	1.50	0.00
51.00	6.39	1.50	0.00
52.00	6.39	1.50	0.00
53.00	6.39	1.50	0.00
54.00	6.39	1.50	0.00
55.00	6.39	1.50	0.00
56.00	6.39	1.50	0.00
57.00	6.39	1.50	0.00
58.00	6.39	1.50	0.00
59.00	6.39	1.50	0.00
60.00	6.39	1.50	0.00
61.00	6.39	1.50	0.00
62.00	6.39	1.50	0.00
63.00	6.39	1.50	0.00
64.00	6.39	1.50	0.00
65.00	6.39	1.50	0.00
66.00	6.39	1.50	0.00
67.00	6.39	1.50	0.00
68.00	6.39	1.50	0.00
69.00	6.39	1.50	0.00
70.00	6.39	1.50	0.00
71.00	6.39	1.50	0.00
72.00	6.39	1.50	0.00

Summary for Subcatchment 18S: North Access Culvert

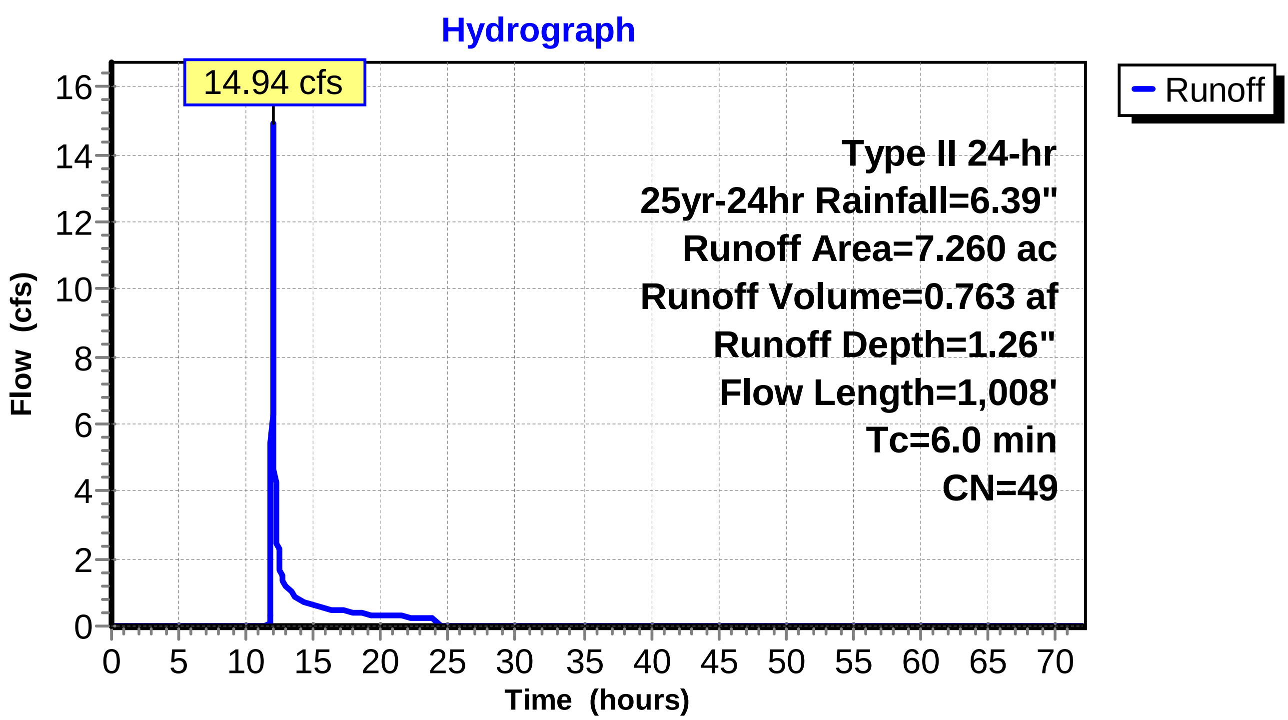
Runoff = 14.94 cfs @ 11.99 hrs, Volume= 0.763 af, Depth= 1.26"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

Area (ac)	CN	Description
7.260	49	50-75% Grass cover, Fair, HSG A
7.260		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
3.6	100	0.5000	0.46		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.2	56	0.5000	4.95		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
2.2	852	0.0362	6.40	42.88	**Channel Flow, Conveyance Channel** Area= 6.7 sf Perim= 9.5' r= 0.71' n= 0.035
6.0	1,008	Total			

Subcatchment 18S: North Access Culvert



Hydrograph for Subcatchment 18S: North Access Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.00
8.00	0.77	0.00	0.00
9.00	0.94	0.00	0.00
10.00	1.16	0.00	0.00
11.00	1.50	0.00	**0.00**
12.00	4.24	0.37	**14.70**
13.00	4.93	0.61	1.17
14.00	5.24	0.74	0.74
15.00	5.45	0.83	0.61
16.00	5.62	0.90	0.49
17.00	5.76	0.96	0.44
18.00	5.89	1.02	0.39
19.00	5.99	1.07	0.35
20.00	6.08	1.11	0.30
21.00	6.16	1.15	0.28
22.00	6.24	1.19	0.28
23.00	6.32	1.23	0.27
24.00	**6.39**	**1.26**	0.26
25.00	6.39	1.26	0.00
26.00	6.39	1.26	0.00
27.00	6.39	1.26	0.00
28.00	6.39	1.26	0.00
29.00	6.39	1.26	0.00
30.00	6.39	1.26	0.00
31.00	6.39	1.26	0.00
32.00	6.39	1.26	0.00
33.00	6.39	1.26	0.00
34.00	6.39	1.26	0.00
35.00	6.39	1.26	0.00
36.00	6.39	1.26	0.00
37.00	6.39	1.26	0.00
38.00	6.39	1.26	0.00
39.00	6.39	1.26	0.00
40.00	6.39	1.26	0.00
41.00	6.39	1.26	0.00
42.00	6.39	1.26	0.00
43.00	6.39	1.26	0.00
44.00	6.39	1.26	0.00
45.00	6.39	1.26	0.00
46.00	6.39	1.26	0.00
47.00	6.39	1.26	0.00
48.00	6.39	1.26	0.00
49.00	6.39	1.26	0.00
50.00	6.39	1.26	0.00
51.00	6.39	1.26	0.00
52.00	6.39	1.26	0.00
53.00	6.39	1.26	0.00
54.00	6.39	1.26	0.00
55.00	6.39	1.26	0.00
56.00	6.39	1.26	0.00
57.00	6.39	1.26	0.00
58.00	6.39	1.26	0.00
59.00	6.39	1.26	0.00
60.00	6.39	1.26	0.00
61.00	6.39	1.26	0.00
62.00	6.39	1.26	0.00
63.00	6.39	1.26	0.00
64.00	6.39	1.26	0.00
65.00	6.39	1.26	0.00
66.00	6.39	1.26	0.00
67.00	6.39	1.26	0.00
68.00	6.39	1.26	0.00
69.00	6.39	1.26	0.00
70.00	6.39	1.26	0.00
71.00	6.39	1.26	0.00
72.00	6.39	1.26	0.00

Summary for Subcatchment 24S: North Sed. Basin Culvert

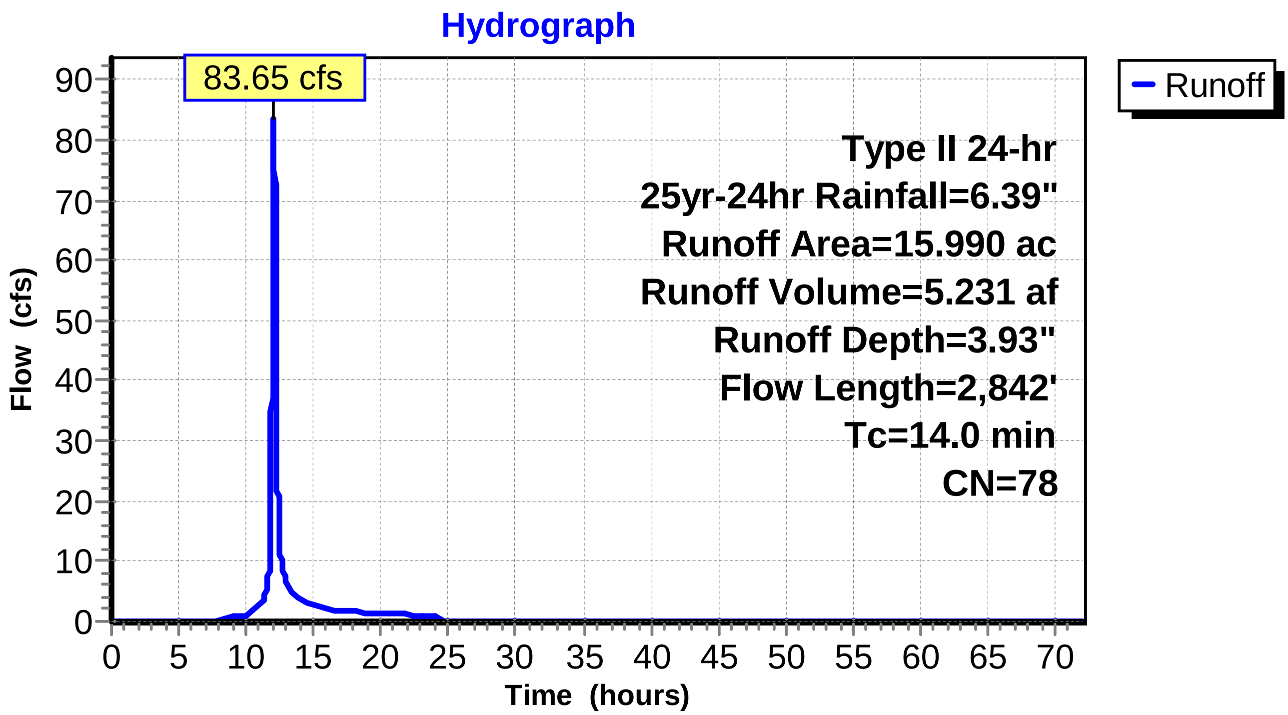
Runoff = 83.65 cfs @ 12.06 hrs, Volume= 5.231 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 24S: North Sed. Basin Culvert



Hydrograph for Subcatchment 24S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)	Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00	54.00	6.39	3.93	0.00
1.00	0.07	0.00	0.00	55.00	6.39	3.93	0.00
2.00	0.14	0.00	0.00	56.00	6.39	3.93	0.00
3.00	0.22	0.00	0.00	57.00	6.39	3.93	0.00
4.00	0.31	0.00	0.00	58.00	6.39	3.93	0.00
5.00	0.40	0.00	0.00	59.00	6.39	3.93	0.00
6.00	0.51	0.00	0.00	60.00	6.39	3.93	0.00
7.00	0.63	0.00	0.06	61.00	6.39	3.93	0.00
8.00	0.77	0.01	0.25	62.00	6.39	3.93	0.00
9.00	0.94	0.04	0.62	63.00	6.39	3.93	0.00
10.00	1.16	0.10	1.12	64.00	6.39	3.93	0.00
11.00	1.50	0.23	2.67	65.00	6.39	3.93	0.00
12.00	4.24	2.08	**73.26**	66.00	6.39	3.93	0.00
13.00	4.93	2.65	**6.29**	67.00	6.39	3.93	0.00
14.00	5.24	2.92	3.64	68.00	6.39	3.93	0.00
15.00	5.45	3.10	2.81	69.00	6.39	3.93	0.00
16.00	5.62	3.25	2.20	70.00	6.39	3.93	0.00
17.00	5.76	3.37	1.90	71.00	6.39	3.93	0.00
18.00	5.89	3.48	1.68	72.00	6.39	3.93	0.00
19.00	5.99	3.57	1.46				
20.00	6.08	3.65	1.23				
21.00	6.16	3.72	1.15				
22.00	6.24	3.79	1.11				
23.00	6.32	3.86	1.07				
24.00	**6.39**	**3.93**	1.02				
25.00	6.39	3.93	0.00				
26.00	6.39	3.93	0.00				
27.00	6.39	3.93	0.00				
28.00	6.39	3.93	0.00				
29.00	6.39	3.93	0.00				
30.00	6.39	3.93	0.00				
31.00	6.39	3.93	0.00				
32.00	6.39	3.93	0.00				
33.00	6.39	3.93	0.00				
34.00	6.39	3.93	0.00				
35.00	6.39	3.93	0.00				
36.00	6.39	3.93	0.00				
37.00	6.39	3.93	0.00				
38.00	6.39	3.93	0.00				
39.00	6.39	3.93	0.00				
40.00	6.39	3.93	0.00				
41.00	6.39	3.93	0.00				
42.00	6.39	3.93	0.00				
43.00	6.39	3.93	0.00				
44.00	6.39	3.93	0.00				
45.00	6.39	3.93	0.00				
46.00	6.39	3.93	0.00				
47.00	6.39	3.93	0.00				
48.00	6.39	3.93	0.00				
49.00	6.39	3.93	0.00				
50.00	6.39	3.93	0.00				
51.00	6.39	3.93	0.00				
52.00	6.39	3.93	0.00				
53.00	6.39	3.93	0.00				

Summary for Subcatchment 25S: South Sed. Basin Culvert

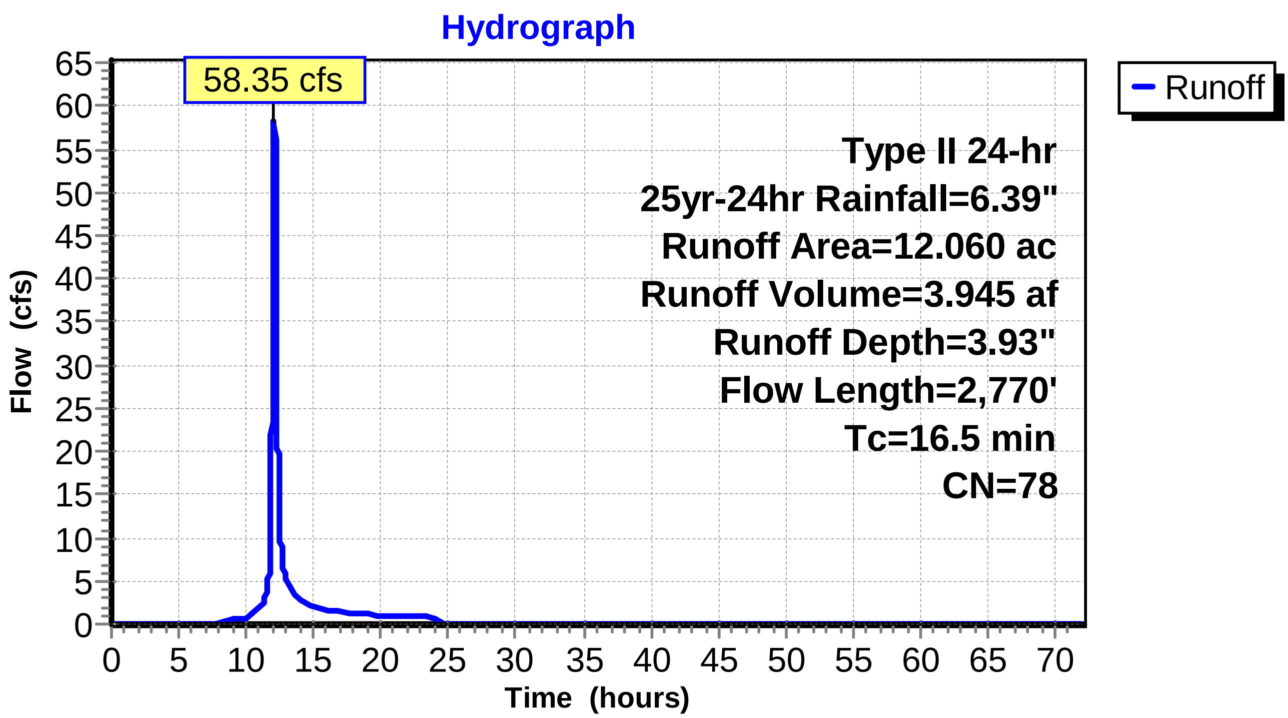
Runoff = 58.35 cfs @ 12.08 hrs, Volume= 3.945 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 25S: South Sed. Basin Culvert



Hydrograph for Subcatchment 25S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.04
8.00	0.77	0.01	0.18
9.00	0.94	0.04	0.45
10.00	1.16	0.10	0.82
11.00	1.50	0.23	1.94
12.00	4.24	2.08	**46.36**
13.00	4.93	2.65	**4.91**
14.00	5.24	2.92	2.80
15.00	5.45	3.10	2.13
16.00	5.62	3.25	1.68
17.00	5.76	3.37	1.44
18.00	5.89	3.48	1.27
19.00	5.99	3.57	1.11
20.00	6.08	3.65	0.94
21.00	6.16	3.72	0.87
22.00	6.24	3.79	0.84
23.00	6.32	3.86	0.81
24.00	**6.39**	**3.93**	0.77
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 26S: Sed. Basin

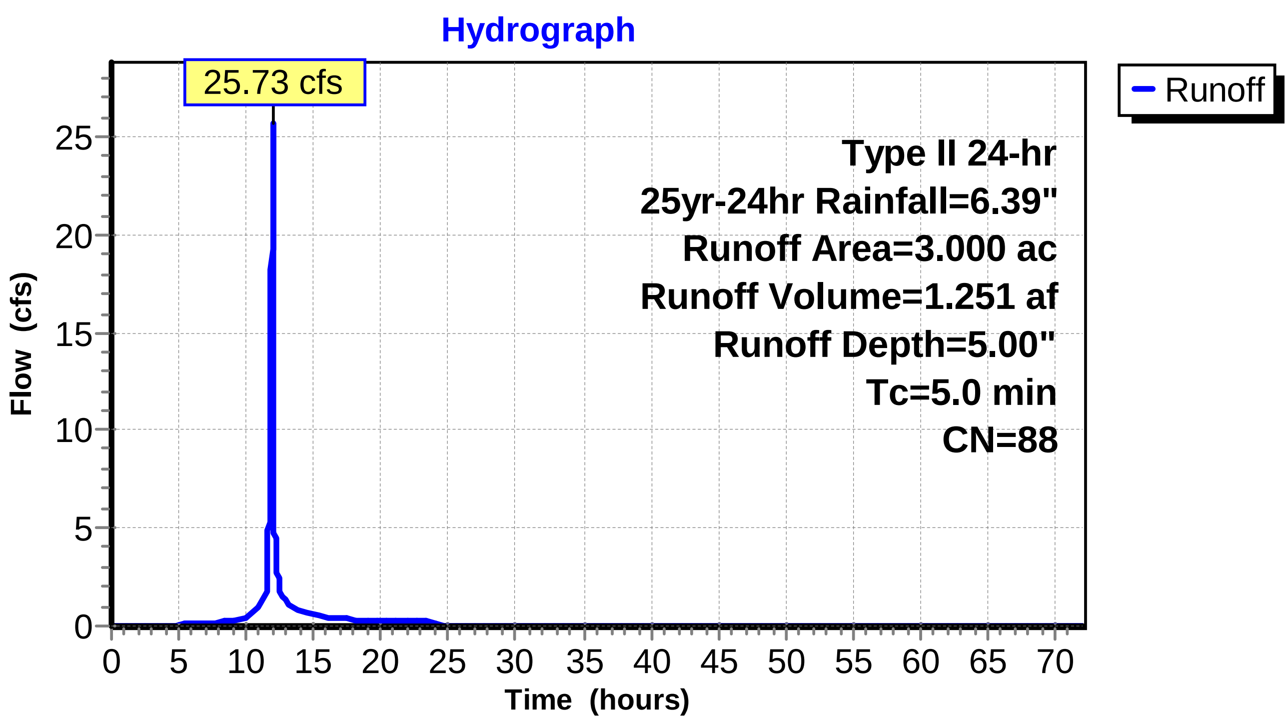
Runoff = 25.73 cfs @ 11.96 hrs, Volume= 1.251 af, Depth= 5.00"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	1.500	98	
*	1.500	78	
	3.000	88	Weighted Average
	1.500		50.00% Pervious Area
	1.500		50.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry, Min. Time of Concentration**

Subcatchment 26S: Sed. Basin



Hydrograph for Subcatchment 26S: Sed. Basin

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)	Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00	54.00	6.39	5.00	0.00
1.00	0.07	0.00	0.00	55.00	6.39	5.00	0.00
2.00	0.14	0.00	0.00	56.00	6.39	5.00	0.00
3.00	0.22	0.00	0.00	57.00	6.39	5.00	0.00
4.00	0.31	0.00	0.01	58.00	6.39	5.00	0.00
5.00	0.40	0.01	0.05	59.00	6.39	5.00	0.00
6.00	0.51	0.04	0.09	60.00	6.39	5.00	0.00
7.00	0.63	0.08	0.14	61.00	6.39	5.00	0.00
8.00	0.77	0.13	0.19	62.00	6.39	5.00	0.00
9.00	0.94	0.22	0.33	63.00	6.39	5.00	0.00
10.00	1.16	0.35	0.47	64.00	6.39	5.00	0.00
11.00	1.50	0.58	**0.95**	65.00	6.39	5.00	0.00
12.00	4.24	2.95	**21.48**	66.00	6.39	5.00	0.00
13.00	4.93	3.61	1.19	67.00	6.39	5.00	0.00
14.00	5.24	3.90	0.71	68.00	6.39	5.00	0.00
15.00	5.45	4.10	0.56	69.00	6.39	5.00	0.00
16.00	5.62	4.26	0.44	70.00	6.39	5.00	0.00
17.00	5.76	4.40	0.38	71.00	6.39	5.00	0.00
18.00	5.89	4.52	0.34	72.00	6.39	5.00	0.00
19.00	5.99	4.62	0.29				
20.00	6.08	4.71	0.25				
21.00	6.16	4.78	0.23				
22.00	6.24	4.86	0.22				
23.00	6.32	4.93	0.22				
24.00	**6.39**	**5.00**	0.21				
25.00	6.39	5.00	0.00				
26.00	6.39	5.00	0.00				
27.00	6.39	5.00	0.00				
28.00	6.39	5.00	0.00				
29.00	6.39	5.00	0.00				
30.00	6.39	5.00	0.00				
31.00	6.39	5.00	0.00				
32.00	6.39	5.00	0.00				
33.00	6.39	5.00	0.00				
34.00	6.39	5.00	0.00				
35.00	6.39	5.00	0.00				
36.00	6.39	5.00	0.00				
37.00	6.39	5.00	0.00				
38.00	6.39	5.00	0.00				
39.00	6.39	5.00	0.00				
40.00	6.39	5.00	0.00				
41.00	6.39	5.00	0.00				
42.00	6.39	5.00	0.00				
43.00	6.39	5.00	0.00				
44.00	6.39	5.00	0.00				
45.00	6.39	5.00	0.00				
46.00	6.39	5.00	0.00				
47.00	6.39	5.00	0.00				
48.00	6.39	5.00	0.00				
49.00	6.39	5.00	0.00				
50.00	6.39	5.00	0.00				
51.00	6.39	5.00	0.00				
52.00	6.39	5.00	0.00				
53.00	6.39	5.00	0.00				

Summary for Pond 17P: North Access Culvert

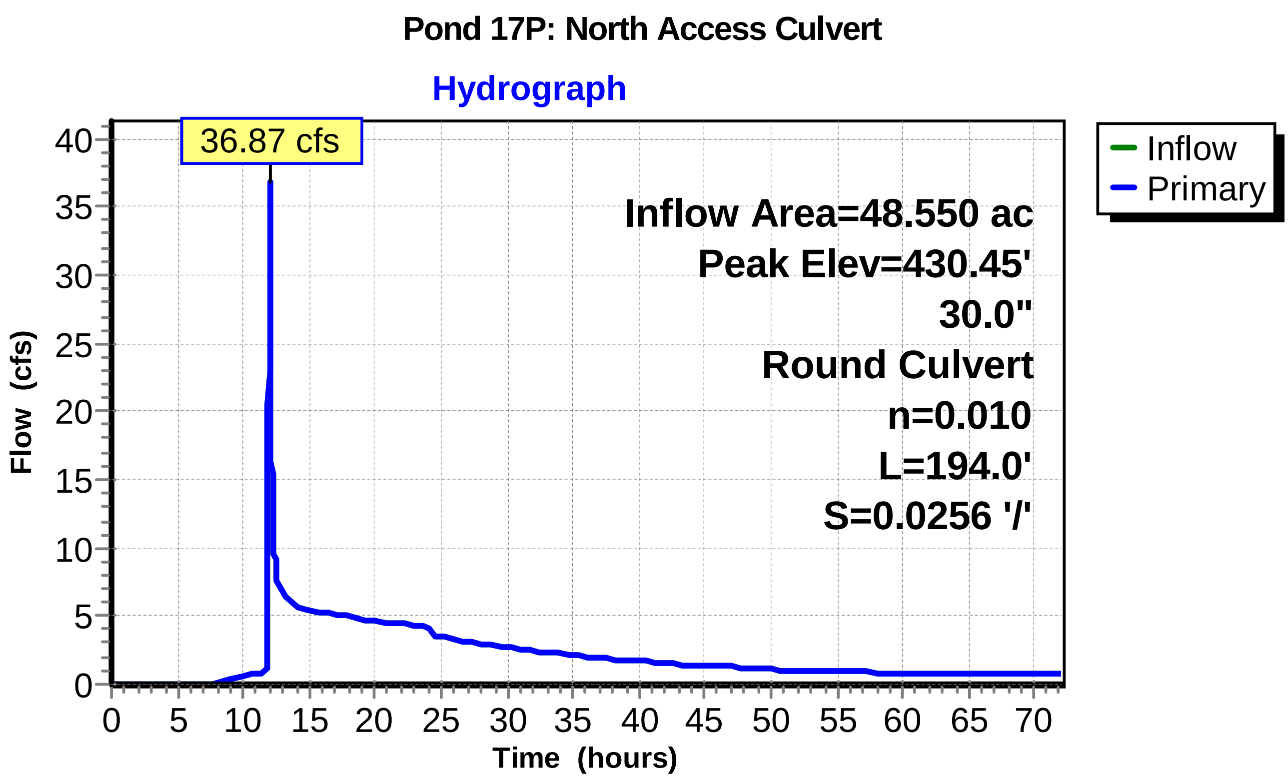
[57] Hint: Peaked at 430.45' (Flood elevation advised)

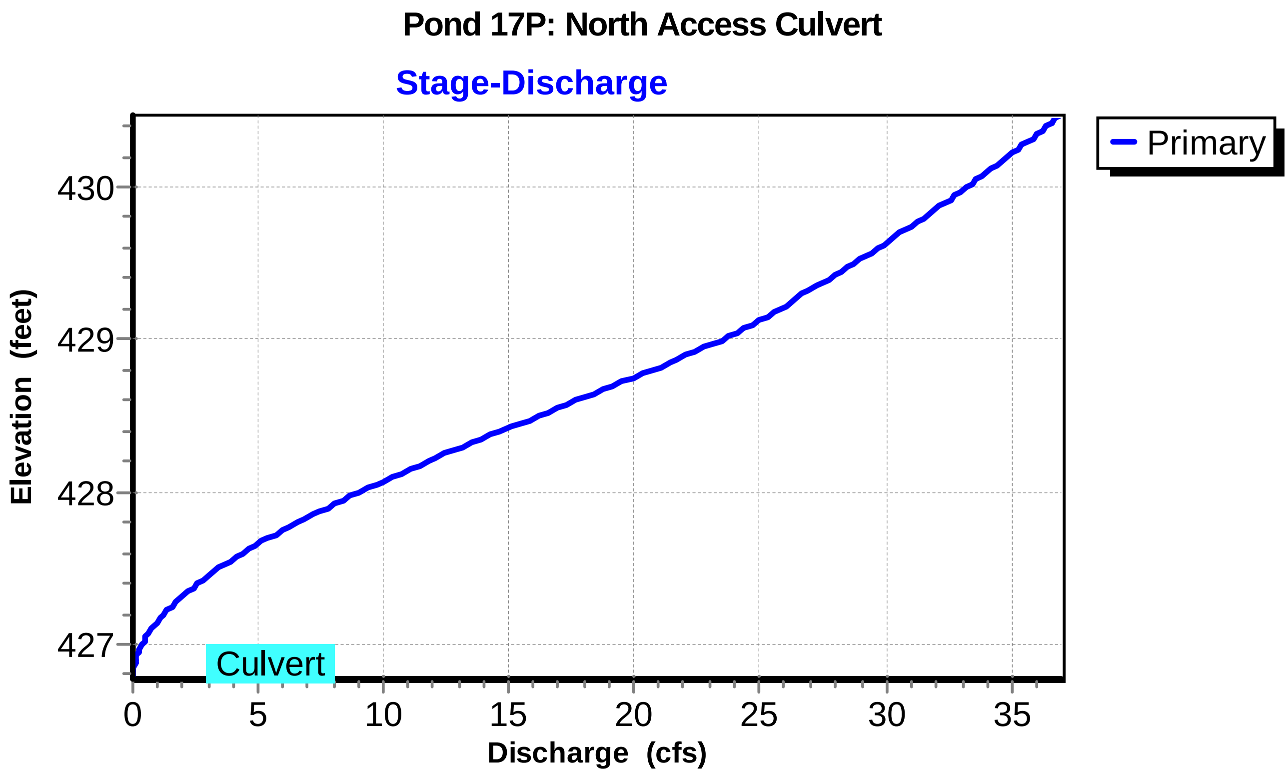
Inflow Area =	48.550 ac,	3.09% Impervious,	Inflow Depth > 2.95"	for 25yr-24hr event
Inflow =	36.87 cfs @	12.01 hrs, Volume=	11.927 af	
Outflow =	36.87 cfs @	12.01 hrs, Volume=	11.927 af, Atten= 0%, Lag= 0.0 min	
Primary =	36.87 cfs @	12.01 hrs, Volume=	11.927 af	

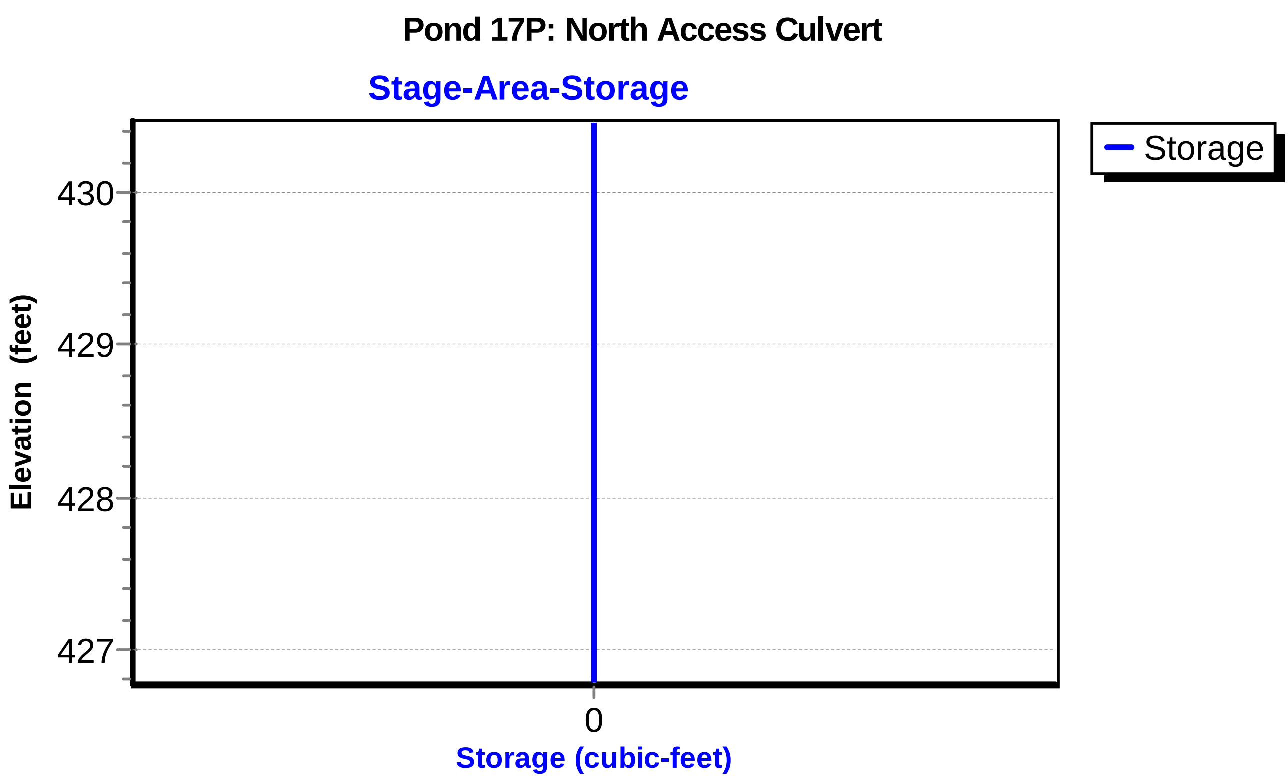
Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Peak Elev= 430.45' @ 12.01 hrs

Device	Routing	Invert	Outlet Devices
#1	Primary	426.77'	**30.0" Round Culvert** L= 194.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 426.77' / 421.80' S= 0.0256 '/' Cc= 0.900 n= 0.010, Flow Area= 4.91 sf

Primary OutFlow Max=36.87 cfs @ 12.01 hrs HW=430.45' (Free Discharge)
└─**1=Culvert** (Inlet Controls 36.87 cfs @ 7.51 fps)




Pond 17P: North Access Culvert
Stage-Discharge
Primary
Culvert
Elevation (feet)
430
429
428
427
0
5
10
15
20
25
30
35
Discharge (cfs)


Pond 17P: North Access Culvert
Stage-Area-Storage
Storage
Elevation (feet)
430
429
428
427
0
Storage (cubic-feet)

Hydrograph for Pond 17P: North Access Culvert

Time (hours)	Inflow (cfs)	Elevation (feet)	Primary (cfs)
0.00	0.00	426.77	0.00
1.00	0.00	426.77	0.00
2.00	0.00	426.77	0.00
3.00	0.00	426.77	0.00
4.00	0.00	426.77	0.00
5.00	0.01	426.80	0.01
6.00	0.02	426.82	0.02
7.00	0.04	426.85	0.04
8.00	0.12	426.90	0.12
9.00	0.30	426.98	0.30
10.00	0.61	427.07	0.61
11.00	0.73	427.10	0.73
12.00	**36.63**	**430.42**	**36.63**
13.00	**6.72**	**427.81**	**6.72**
14.00	5.78	427.73	5.78
15.00	5.51	427.70	5.51
16.00	5.23	427.68	5.23
17.00	5.09	427.67	5.09
18.00	4.95	427.65	4.95
19.00	4.79	427.64	4.79
20.00	4.61	427.62	4.61
21.00	4.51	427.61	4.51
22.00	4.43	427.60	4.43
23.00	4.34	427.59	4.34
24.00	4.25	427.58	4.25
25.00	3.44	427.50	3.44
26.00	3.22	427.47	3.22
27.00	3.09	427.46	3.09
28.00	2.97	427.44	2.97
29.00	2.84	427.43	2.84
30.00	2.69	427.41	2.69
31.00	2.53	427.39	2.53
32.00	2.43	427.38	2.43
33.00	2.34	427.36	2.34
34.00	2.25	427.35	2.25
35.00	2.14	427.34	2.14
36.00	2.00	427.32	2.00
37.00	1.91	427.30	1.91
38.00	1.85	427.29	1.85
39.00	1.78	427.28	1.78
40.00	1.71	427.27	1.71
41.00	1.64	427.26	1.64
42.00	1.54	427.25	1.54
43.00	1.44	427.23	1.44
44.00	1.40	427.22	1.40
45.00	1.36	427.22	1.36
46.00	1.32	427.21	1.32
47.00	1.28	427.20	1.28
48.00	1.23	427.20	1.23
49.00	1.18	427.19	1.18
50.00	1.11	427.17	1.11
51.00	1.04	427.16	1.04
52.00	1.01	427.15	1.01
53.00	0.99	427.15	0.99
54.00	0.97	427.15	0.97
55.00	0.95	427.14	0.95
56.00	0.93	427.14	0.93
57.00	0.90	427.13	0.90
58.00	0.87	427.13	0.87
59.00	0.83	427.12	0.83
60.00	0.79	427.11	0.79
61.00	0.74	427.10	0.74
62.00	0.73	427.10	0.73
63.00	0.73	427.10	0.73
64.00	0.73	427.10	0.73
65.00	0.73	427.10	0.73
66.00	0.73	427.10	0.73
67.00	0.73	427.10	0.73
68.00	0.73	427.10	0.73
69.00	0.73	427.10	0.73
70.00	0.73	427.10	0.73
71.00	0.73	427.10	0.73
72.00	0.73	427.10	0.73

Stage-Discharge for Pond 17P: North Access Culvert

Elevation (feet)	Primary (cfs)	Elevation (feet)	Primary (cfs)	Elevation (feet)	Primary (cfs)	Elevation (feet)	Primary (cfs)
426.77	0.00	427.85	7.19	428.93	22.56	430.01	33.34
426.79	0.00	427.87	7.43	428.95	22.83	430.03	33.51
426.81	0.01	427.89	7.67	428.97	23.10	430.05	33.68
426.83	0.03	427.91	7.92	428.99	23.37	430.07	33.84
426.85	0.05	427.93	8.18	429.01	23.63	430.09	34.01
426.87	0.07	427.95	8.43	429.03	23.89	430.11	34.17
426.89	0.10	427.97	8.69	429.05	24.15	430.13	34.33
426.91	0.14	427.99	8.95	429.07	24.40	430.15	34.49
426.93	0.18	428.01	9.21	429.09	24.64	430.17	34.66
426.95	0.23	428.03	9.48	429.11	24.88	430.19	34.82
426.97	0.28	428.05	9.74	429.13	25.11	430.21	34.98
426.99	0.34	428.07	10.01	429.15	25.33	430.23	35.14
427.01	0.40	428.09	10.28	429.17	25.54	430.25	35.30
427.03	0.47	428.11	10.56	429.19	25.75	430.27	35.45
427.05	0.54	428.13	10.84	429.21	25.94	430.29	35.61
427.07	0.62	428.15	11.11	429.23	26.12	430.31	35.77
427.09	0.71	428.17	11.39	429.25	26.29	430.33	35.92
427.11	0.80	428.19	11.68	429.27	26.43	430.35	36.08
427.13	0.89	428.21	11.96	429.29	26.64	430.37	36.23
427.15	0.99	428.23	12.25	429.31	26.84	430.39	36.39
427.17	1.09	428.25	12.53	429.33	27.05	430.41	36.54
427.19	1.20	428.27	12.82	429.35	27.26	430.43	36.69
427.21	1.31	428.29	13.11	429.37	27.46	430.45	36.84
427.23	1.43	428.31	13.41	429.39	27.66	430.47	**37.00**
427.25	1.55	428.33	13.70	429.41	27.87		
427.27	1.68	428.35	13.99	429.43	28.07		
427.29	1.81	428.37	14.29	429.45	28.26		
427.31	1.95	428.39	14.59	429.47	28.46		
427.33	2.09	428.41	14.88	429.49	28.66		
427.35	2.24	428.43	15.18	429.51	28.85		
427.37	2.39	428.45	15.48	429.53	29.04		
427.39	2.54	428.47	15.78	429.55	29.24		
427.41	2.70	428.49	16.08	429.57	29.43		
427.43	2.87	428.51	16.38	429.59	29.62		
427.45	3.03	428.53	16.68	429.61	29.80		
427.47	3.21	428.55	16.98	429.63	29.99		
427.49	3.38	428.57	17.28	429.65	30.18		
427.51	3.56	428.59	17.58	429.67	30.36		
427.53	3.74	428.61	17.89	429.69	30.54		
427.55	3.93	428.63	18.19	429.71	30.73		
427.57	4.12	428.65	18.49	429.73	30.91		
427.59	4.32	428.67	18.79	429.75	31.09		
427.61	4.52	428.69	19.08	429.77	31.27		
427.63	4.72	428.71	19.38	429.79	31.44		
427.65	4.93	428.73	19.68	429.81	31.62		
427.67	5.14	428.75	19.98	429.83	31.80		
427.69	5.35	428.77	20.27	429.85	31.97		
427.71	5.57	428.79	20.56	429.87	32.15		
427.73	5.79	428.81	20.86	429.89	32.32		
427.75	6.02	428.83	21.14	429.91	32.49		
427.77	6.24	428.85	21.43	429.93	32.66		
427.79	6.47	428.87	21.72	429.95	32.84		
427.81	6.71	428.89	22.00	429.97	33.01		
427.83	6.94	428.91	22.28	429.99	33.17		

Stage-Area-Storage for Pond 17P: North Access Culvert

Elevation (feet)	Storage (cubic-feet)	Elevation (feet)	Storage (cubic-feet)
426.77	**0**	429.47	0
426.82	0	429.52	0
426.87	0	429.57	0
426.92	0	429.62	0
426.97	0	429.67	0
427.02	0	429.72	0
427.07	0	429.77	0
427.12	0	429.82	0
427.17	0	429.87	0
427.22	0	429.92	0
427.27	0	429.97	0
427.32	0	430.02	0
427.37	0	430.07	0
427.42	0	430.12	0
427.47	0	430.17	0
427.52	0	430.22	0
427.57	0	430.27	0
427.62	0	430.32	0
427.67	0	430.37	0
427.72	0	430.42	0
427.77	0	430.47	0
427.82	0		
427.87	0		
427.92	0		
427.97	0		
428.02	0		
428.07	0		
428.12	0		
428.17	0		
428.22	0		
428.27	0		
428.32	0		
428.37	0		
428.42	0		
428.47	0		
428.52	0		
428.57	0		
428.62	0		
428.67	0		
428.72	0		
428.77	0		
428.82	0		
428.87	0		
428.92	0		
428.97	0		
429.02	0		
429.07	0		
429.12	0		
429.17	0		
429.22	0		
429.27	0		
429.32	0		
429.37	0		
429.42	0		

Summary for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Inflow Area = 31.050 ac, 4.83% Impervious, Inflow Depth = 4.03" for 25yr-24hr event
Inflow = 150.51 cfs @ 12.05 hrs, Volume= 10.426 af
Outflow = 3.96 cfs @ 16.43 hrs, Volume= 9.885 af, Atten= 97%, Lag= 263.0 min
Primary = 3.96 cfs @ 16.43 hrs, Volume= 9.885 af
Secondary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Stor-Ind method, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Starting Elev= 439.25' Surf.Area= 0.512 ac Storage= 1.425 af
Peak Elev= 447.32' @ 16.43 hrs Surf.Area= 1.269 ac Storage= 8.622 af (7.197 af above start)
Flood Elev= 450.50' Surf.Area= 1.540 ac Storage= 13.095 af (11.670 af above start)

Plug-Flow detention time= 1,350.0 min calculated for 8.459 af (81% of inflow)
Center-of-Mass det. time= 1,087.9 min (1,905.4 - 817.5)

Volume	Invert	Avail.Storage	Storage Description
#1	434.00'	14.687 af	**Sediment Basin (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
434.00	0.030	0.000	0.000
435.00	0.130	0.080	0.080
436.00	0.220	0.175	0.255
437.00	0.300	0.260	0.515
438.00	0.390	0.345	0.860
439.00	0.490	0.440	1.300
440.00	0.580	0.535	1.835
441.00	0.670	0.625	2.460
442.00	0.770	0.720	3.180
443.00	0.870	0.820	4.000
444.00	0.960	0.915	4.915
445.00	1.060	1.010	5.925
446.00	1.150	1.105	7.030
447.00	1.240	1.195	8.225
448.00	1.330	1.285	9.510
449.00	1.410	1.370	10.880
450.00	1.500	1.455	12.335
451.00	1.580	1.540	13.875
451.50	1.670	0.812	14.687

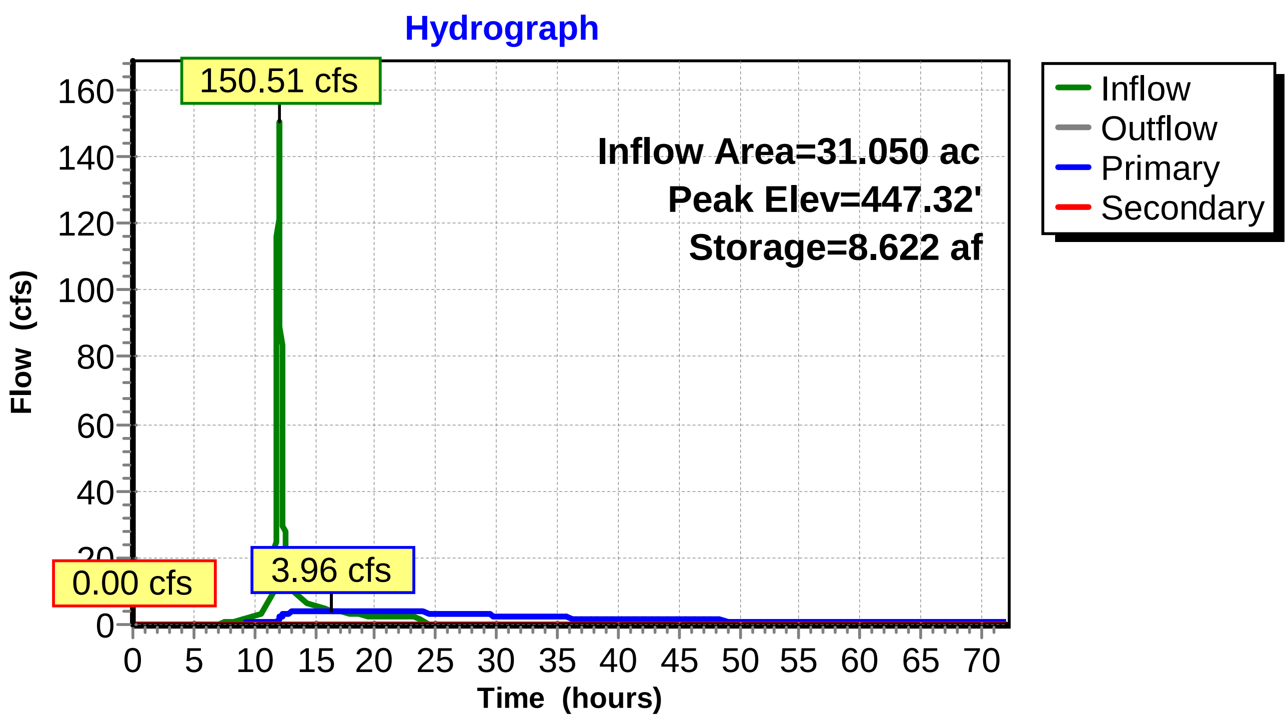
Device	Routing	Invert	Outlet Devices
#1	Primary	438.75'	**18.0" Round RCP_Round 18"** L= 100.0' RCP, square edge headwall, Ke= 0.500 Inlet / Outlet Invert= 438.75' / 436.75' S= 0.0200 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 1.77 sf
#2	Device 1	439.25'	**0.734 cfs Constant Flow/Skimmer** Phase-In= 0.50'
#3	Device 1	441.25'	**1.5" Vert. Orifice/Grate X 5.00 columns** X 8 rows with 12.0" cc spacing C= 0.600
#4	Device 1	450.00'	**48.0" x 48.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

#5	Secondary	450.25'	**25.0' long x 10.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.49 2.56 2.70 2.69 2.68 2.69 2.67 2.64

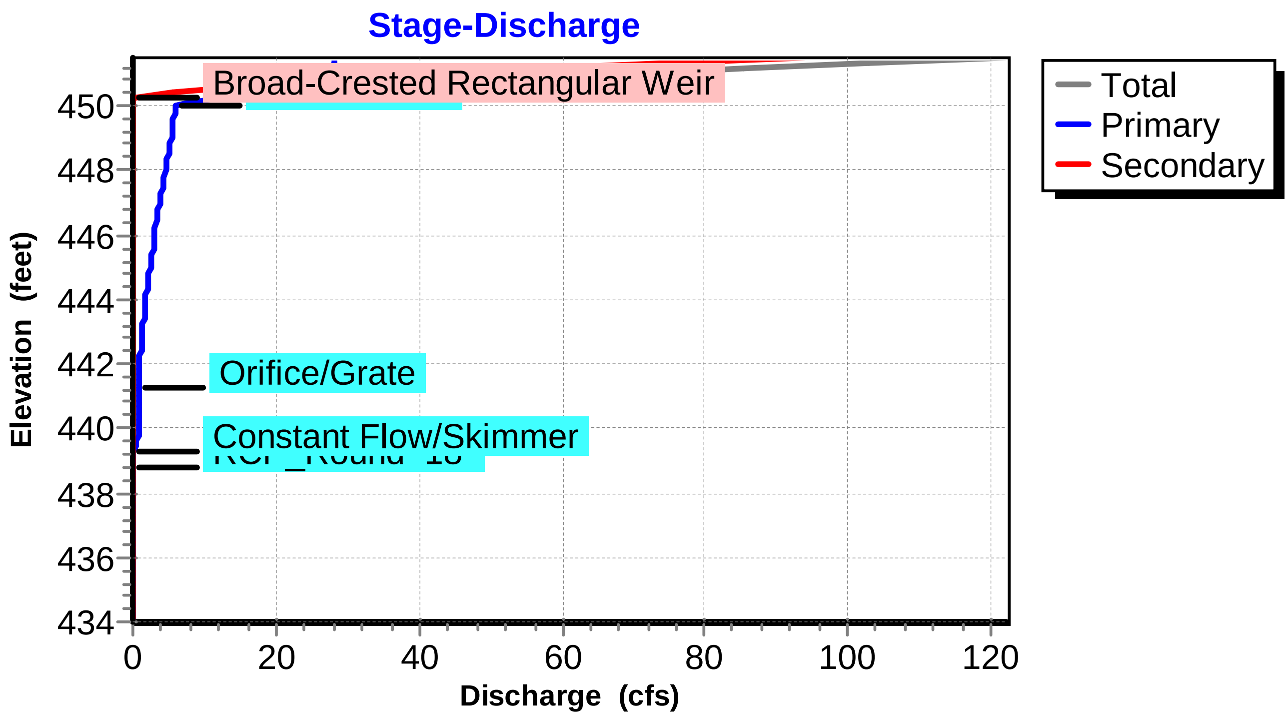
Primary OutFlow Max=3.97 cfs @ 16.43 hrs HW=447.32' (Free Discharge)
1=RCP_Round 18" (Passes 3.97 cfs of 23.40 cfs potential flow)
2=Constant Flow/Skimmer (Constant Controls 0.73 cfs)
3=Orifice/Grate (Orifice Controls 3.23 cfs @ 8.05 fps)
4=Orifice/Grate (Controls 0.00 cfs)

Secondary OutFlow Max=0.00 cfs @ 0.00 hrs HW=439.25' (Free Discharge)
5=Broad-Crested Rectangular Weir (Controls 0.00 cfs)

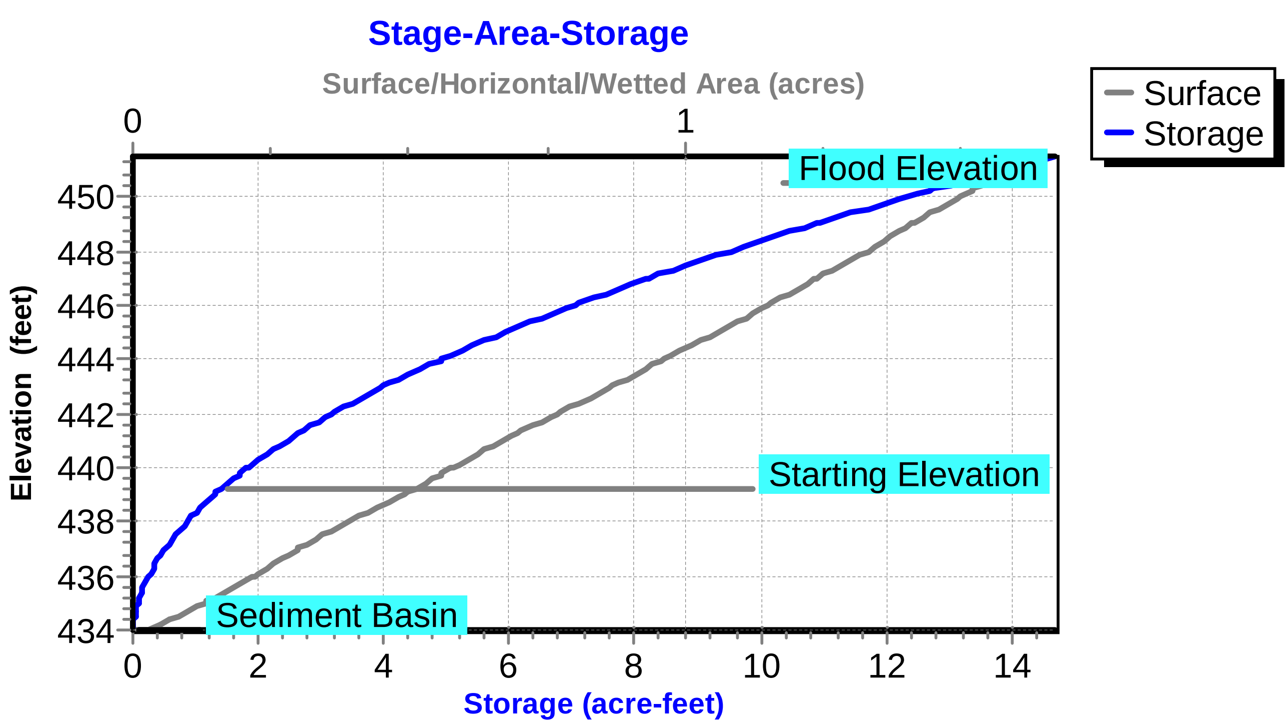
Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Pond 20P: Sediment Pond (WSE 439.25) Full Operation



Hydrograph for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Time (hours)	Inflow (cfs)	Storage (acre-feet)	Elevation (feet)	Outflow (cfs)	Primary (cfs)	Secondary (cfs)
0.00	0.00	1.425	439.25	0.00	0.00	**0.00**
2.00	0.00	1.425	439.25	0.00	0.00	0.00
4.00	0.01	1.425	439.25	0.00	0.00	0.00
6.00	0.09	1.432	439.26	0.02	0.02	0.00
8.00	0.62	1.470	439.34	0.12	0.12	0.00
10.00	2.40	1.646	439.67	0.61	0.61	0.00
12.00	**141.10**	4.112	443.13	1.40	1.40	0.00
14.00	**7.16**	8.347	447.10	3.82	3.82	0.00
16.00	4.31	**8.617**	**447.31**	**3.96**	**3.96**	0.00
18.00	3.29	**8.580**	**447.28**	**3.93**	**3.93**	0.00
20.00	2.42	8.409	447.15	3.85	3.85	0.00
22.00	2.17	8.157	446.94	3.73	3.73	0.00
24.00	2.00	7.897	446.73	3.59	3.59	0.00
26.00	0.00	7.366	446.29	3.22	3.22	0.00
28.00	0.00	6.855	445.85	2.97	2.97	0.00
30.00	0.00	6.386	445.43	2.69	2.69	0.00
32.00	0.00	5.966	445.04	2.43	2.43	0.00
34.00	0.00	5.579	444.67	2.25	2.25	0.00
36.00	0.00	5.227	444.32	2.00	2.00	0.00
38.00	0.00	4.910	444.00	1.85	1.85	0.00
40.00	0.00	4.616	443.68	1.71	1.71	0.00
42.00	0.00	4.346	443.39	1.54	1.54	0.00
44.00	0.00	4.106	443.12	1.40	1.40	0.00
46.00	0.00	3.882	442.86	1.32	1.32	0.00
48.00	0.00	3.670	442.61	1.23	1.23	0.00
50.00	0.00	3.476	442.38	1.11	1.11	0.00
52.00	0.00	3.303	442.16	1.01	1.01	0.00
54.00	0.00	3.140	441.95	0.97	0.97	0.00
56.00	0.00	2.983	441.74	0.93	0.93	0.00
58.00	0.00	2.834	441.54	0.87	0.87	0.00
60.00	0.00	2.697	441.34	0.79	0.79	0.00
62.00	0.00	2.573	441.17	0.73	0.73	0.00
64.00	0.00	2.452	440.99	0.73	0.73	0.00
66.00	0.00	2.331	440.80	0.73	0.73	0.00
68.00	0.00	2.209	440.62	0.73	0.73	0.00
70.00	0.00	2.088	440.42	0.73	0.73	0.00
72.00	0.00	1.967	440.22	0.73	0.73	0.00

Stage-Discharge for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Elevation (feet)	Discharge (cfs)	Primary (cfs)	Secondary (cfs)
434.00	0.00	0.00	0.00
434.20	0.00	0.00	0.00
434.40	0.00	0.00	0.00
434.60	0.00	0.00	0.00
434.80	0.00	0.00	0.00
435.00	0.00	0.00	0.00
435.20	0.00	0.00	0.00
435.40	0.00	0.00	0.00
435.60	0.00	0.00	0.00
435.80	0.00	0.00	0.00
436.00	0.00	0.00	0.00
436.20	0.00	0.00	0.00
436.40	0.00	0.00	0.00
436.60	0.00	0.00	0.00
436.80	0.00	0.00	0.00
437.00	0.00	0.00	0.00
437.20	0.00	0.00	0.00
437.40	0.00	0.00	0.00
437.60	0.00	0.00	0.00
437.80	0.00	0.00	0.00
438.00	0.00	0.00	0.00
438.20	0.00	0.00	0.00
438.40	0.00	0.00	0.00
438.60	0.00	0.00	0.00
438.80	0.00	0.00	0.00
439.00	0.00	0.00	0.00
439.20	0.00	0.00	0.00
439.40	0.22	0.22	0.00
439.60	0.51	0.51	0.00
439.80	0.73	0.73	0.00
440.00	0.73	0.73	0.00
440.20	0.73	0.73	0.00
440.40	0.73	0.73	0.00
440.60	0.73	0.73	0.00
440.80	0.73	0.73	0.00
441.00	0.73	0.73	0.00
441.20	0.73	0.73	0.00
441.40	0.82	0.82	0.00
441.60	0.89	0.89	0.00
441.80	0.94	0.94	0.00
442.00	0.98	0.98	0.00
442.20	1.01	1.01	0.00
442.40	1.13	1.13	0.00
442.60	1.23	1.23	0.00
442.80	1.30	1.30	0.00
443.00	1.36	1.36	0.00
443.20	1.42	1.42	0.00
443.40	1.56	1.56	0.00
443.60	1.67	1.67	0.00
443.80	1.77	1.77	0.00
444.00	1.85	1.85	0.00
444.20	1.92	1.92	0.00
444.40	2.08	2.08	0.00
444.60	2.21	2.21	0.00
444.80	2.32	2.32	0.00
445.00	2.41	2.41	0.00
445.20	2.50	2.50	0.00
445.40	2.67	2.67	0.00
445.60	2.82	2.82	0.00
445.80	2.94	2.94	0.00
446.00	3.05	3.05	0.00
446.20	3.16	3.16	0.00
446.40	3.34	3.34	0.00
446.60	3.50	3.50	0.00
446.80	3.64	3.64	0.00
447.00	3.76	3.76	0.00
447.20	3.87	3.87	0.00
447.40	4.07	4.07	0.00
447.60	4.24	4.24	0.00
447.80	4.39	4.39	0.00
448.00	4.52	4.52	0.00
448.20	4.65	4.65	0.00
448.40	4.85	4.85	0.00
448.60	5.04	5.04	0.00
448.80	5.20	5.20	0.00
449.00	5.34	5.34	0.00
449.20	5.48	5.48	0.00
449.40	5.61	5.61	0.00
449.60	5.73	5.73	0.00
449.80	5.85	5.85	0.00
450.00	5.97	5.97	0.00
450.20	10.76	10.76	0.00
450.40	23.04	19.43	3.62
450.60	40.48	27.32	13.16
450.80	54.71	27.54	27.18
451.00	71.47	27.75	43.72
451.20	90.07	27.97	62.10
451.40	**111.04**	**28.19**	**82.86**

Stage-Area-Storage for Pond 20P: Sediment Pond (WSE 439.25) Full Operation

Elevation (feet)	Surface (acres)	Storage (acre-feet)	Elevation (feet)	Surface (acres)	Storage (acre-feet)
434.00	0.030	0.000	444.80	1.040	5.715
434.20	0.050	0.008	445.00	1.060	5.925
434.40	0.070	0.020	445.20	1.078	6.139
434.60	0.090	0.036	445.40	1.096	6.356
434.80	0.110	0.056	445.60	1.114	6.577
435.00	0.130	0.080	445.80	1.132	6.802
435.20	0.148	0.108	446.00	1.150	7.030
435.40	0.166	0.139	446.20	1.168	7.262
435.60	0.184	0.174	446.40	1.186	7.497
435.80	0.202	0.213	446.60	1.204	7.736
436.00	0.220	0.255	446.80	1.222	7.979
436.20	0.236	0.301	447.00	1.240	8.225
436.40	0.252	0.349	447.20	1.258	8.475
436.60	0.268	0.401	447.40	1.276	8.728
436.80	0.284	0.457	447.60	1.294	8.985
437.00	0.300	0.515	447.80	1.312	9.246
437.20	0.318	0.577	448.00	1.330	9.510
437.40	0.336	0.642	448.20	1.346	9.778
437.60	0.354	0.711	448.40	1.362	10.048
437.80	0.372	0.784	448.60	1.378	10.322
438.00	0.390	0.860	448.80	1.394	10.600
438.20	0.410	0.940	449.00	1.410	10.880
438.40	0.430	1.024	449.20	1.428	11.164
438.60	0.450	1.112	449.40	1.446	11.451
438.80	0.470	1.204	449.60	1.464	11.742
439.00	0.490	1.300	449.80	1.482	12.037
439.20	0.508	1.400	450.00	1.500	12.335
439.40	0.526	1.503	450.20	1.516	12.637
439.60	0.544	1.610	450.40	1.532	12.941
439.80	0.562	1.721	450.60	1.548	13.249
440.00	0.580	1.835	450.80	1.564	13.561
440.20	0.598	1.953	451.00	1.580	13.875
440.40	0.616	2.074	451.20	1.616	14.195
440.60	0.634	2.199	451.40	**1.652**	**14.521**
440.80	0.652	2.328			
441.00	0.670	2.460			
441.20	0.690	2.596			
441.40	0.710	2.736			
441.60	0.730	2.880			
441.80	0.750	3.028			
442.00	0.770	3.180			
442.20	0.790	3.336			
442.40	0.810	3.496			
442.60	0.830	3.660			
442.80	0.850	3.828			
443.00	0.870	4.000			
443.20	0.888	4.176			
443.40	0.906	4.355			
443.60	0.924	4.538			
443.80	0.942	4.725			
444.00	0.960	4.915			
444.20	0.980	5.109			
444.40	1.000	5.307			
444.60	1.020	5.509			

HY-8 Culvert Analysis Report

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 36.87 cfs

Design Flow: 36.87 cfs

Maximum Flow: 36.87 cfs

Table 1 - Summary of Culvert Flows at Crossing: North Access Road

Headwater Elevation (ft)	Total Discharge (cfs)	1-30in RCP Discharge (cfs)	Roadway Discharge (cfs)	Iterations
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
428.70	36.87	36.87	0.00	1
451.50	120.15	120.15	0.00	Overtopping

Rating Curve Plot for Crossing: North Access Road

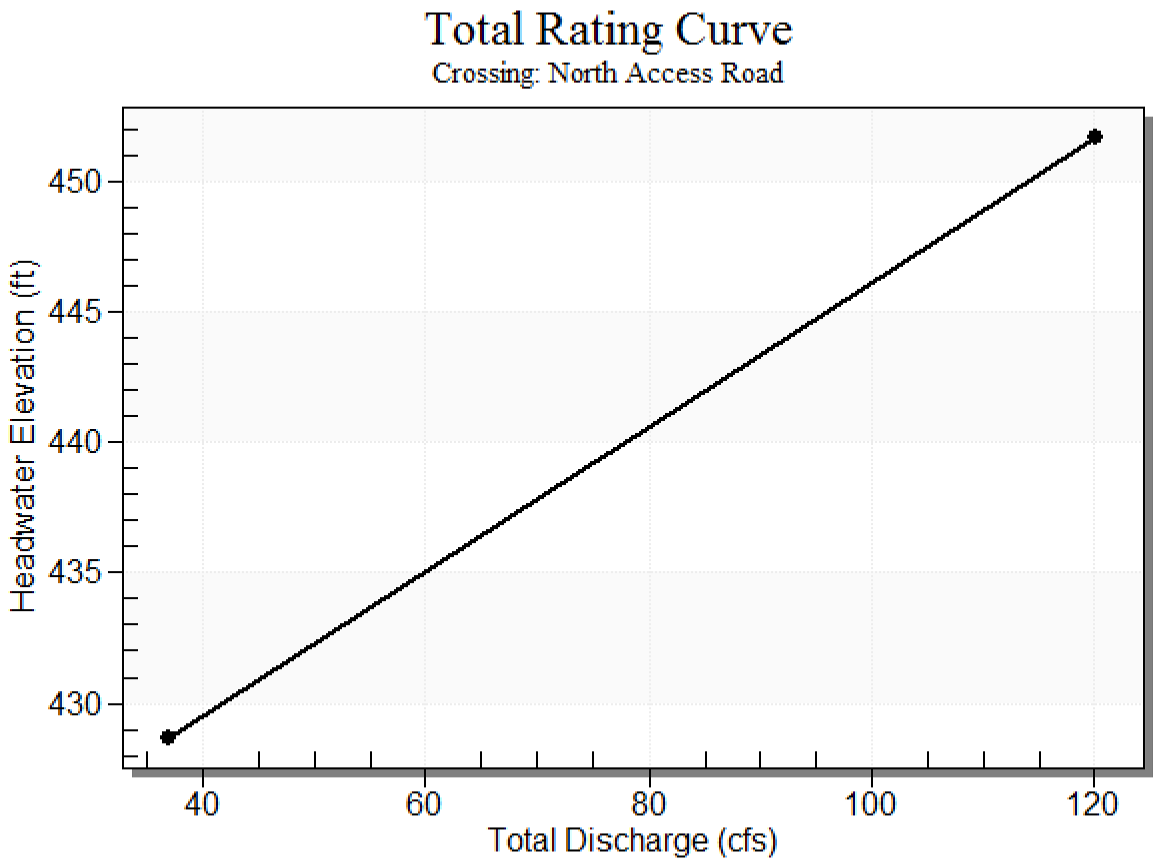


Table 2 - Culvert Summary Table: 1-30in RCP

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922
36.87	36.87	428.70	3.878	1.938	5-S2n	1.351	2.055	1.445	1.332	12.152	6.922

****

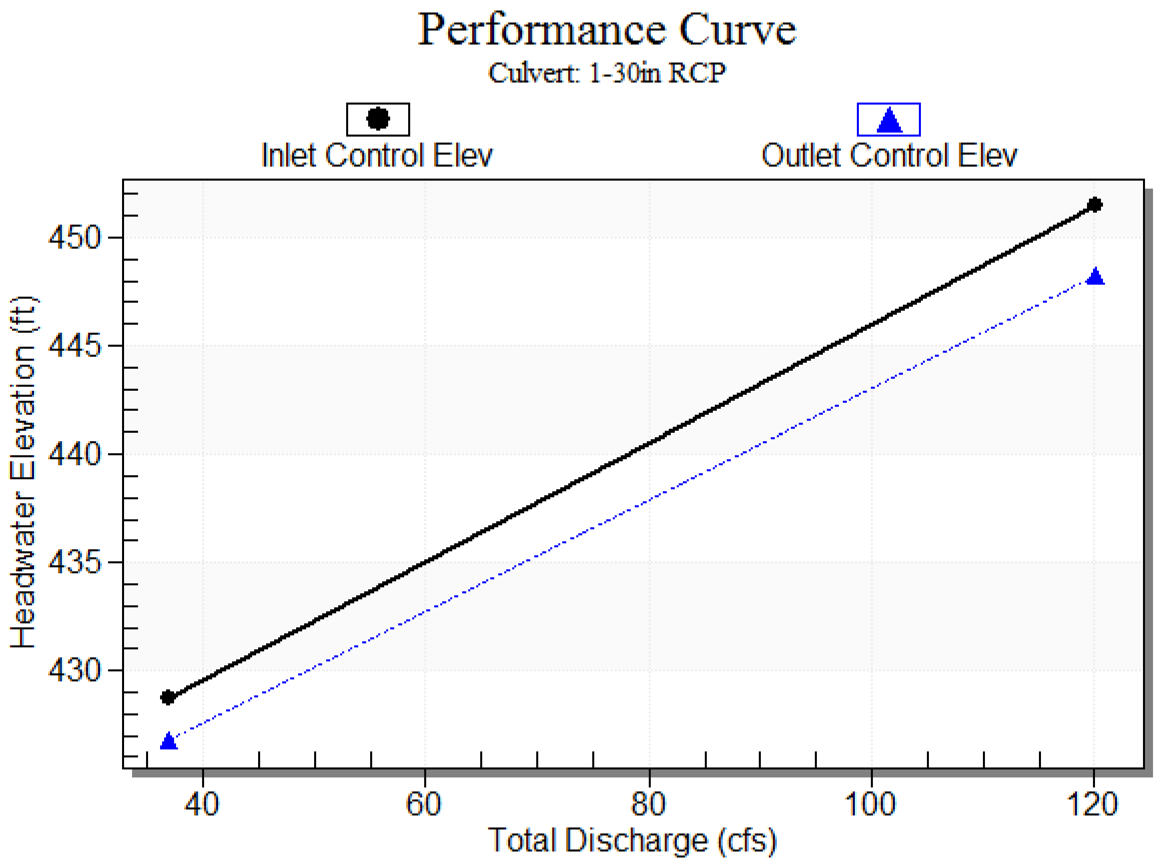
Straight Culvert

Inlet Elevation (invert): 424.82 ft,   Outlet Elevation (invert): 422.27 ft

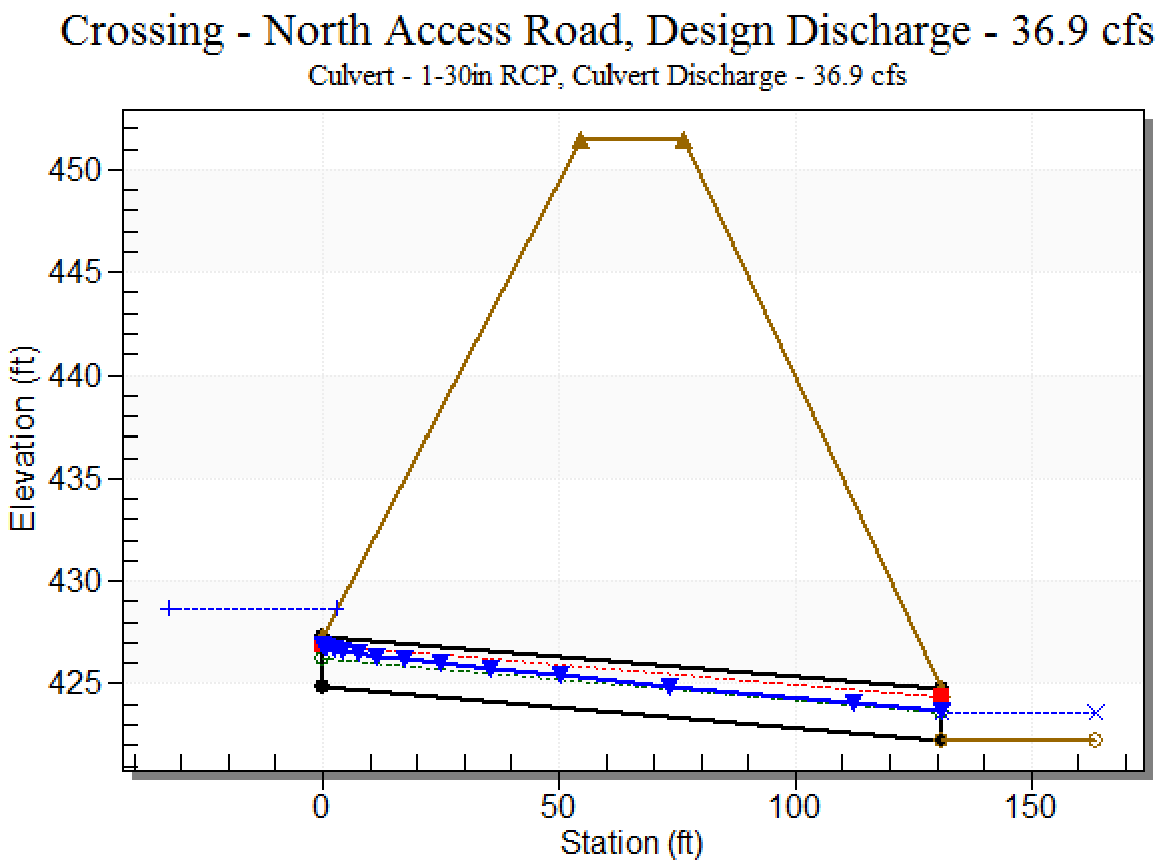
Culvert Length: 131.02 ft,   Culvert Slope: 0.0195

****

Culvert Performance Curve Plot: 1-30in RCP



Water Surface Profile Plot for Culvert: 1-30in RCP



Site Data - 1-30in RCP

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 424.82 ft

Outlet Station: 131.00 ft

Outlet Elevation: 422.27 ft

Number of Barrels: 1

Culvert Data Summary - 1-30in RCP

Barrel Shape: Circular

Barrel Diameter: 2.50 ft

Barrel Material: Concrete

Embedment: 0.00 in

Barrel Manning's n: 0.0120

Culvert Type: Straight

Inlet Configuration: Square Edge with Headwall

Inlet Depression: None

Table 3 - Downstream Channel Rating Curve (Crossing: North Access Road)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)	Velocity (ft/s)	Shear (psf)	Froude Number
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49
36.87	423.60	1.33	6.92	4.07	1.49

Tailwater Channel Data - North Access Road

Tailwater Channel Option: Triangular Channel

Side Slope (H:V): 3.00 (_:1)

Channel Slope: 0.0490

Channel Manning's n: 0.0350

Channel Invert Elevation: 422.27 ft

Roadway Data for Crossing: North Access Road

Roadway Profile Shape: Constant Roadway Elevation

Crest Length: 300.00 ft

Crest Elevation: 451.50 ft

Roadway Surface: Gravel

Roadway Top Width: 22.00 ft

APPENDIX F.1C

Perimeter Channel Design(Rev 3)

I. PURPOSE

The purpose of this analysis is to design the proposed perimeter ditch for the Scepter Class II Disposal Facility in Waverly, TN consistent with state regulations.

II. SITE AND PROJECT DESCRIPTION

The channel design was performed as part of the Part II Solid Waste Permit Application. The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management. Refer to Exhibit 1 for a depiction of the channel locations in relation to the landfill.

The following sections summarize the design criteria, procedure, assumptions, and results of the channels design.

III. REGULATORY REQUIREMENTS / DESIGN CRITERIA

The below parts of the TDEC regulations specify requirements for the design of the channels:

Rule 0400-11-01-.04(2)(i)1.

The operator must design, construct, operate, and maintain a run-on control system capable of preventing flow onto the active portion of the facility for all flow up to and including peak discharge from a 24-hour, 25-year storm.

Rule 0400-11-01-.04(2)(i)2.

The operator must design, construct, operate, and maintain a run-off management system to collect and control at least the peak flow resulting from a 24-hour, 25 year storm.

The results of the analysis presented herein show that the channels are designed to collect and control at least the peak flow resulting from a 25-year/24-hour storm. The perimeter ditch will convey landfill run-off flow towards the sediment basin. The landfill perimeter follows the sites existing high point ridge line, therefore run-on flows are absent.

Rule 0400-11-01-.04(2)(i)6.

The operator must take other erosion control measure (e.g., temporary mulching or seeding, silt barriers) as necessary to control erosion of the site.

Proper channel lining will be selected based on anticipated velocities such that erosion within the channels will be significantly reduced.

IV. PROCEDURE

Design of the landfill site stormwater features was an iterative process beginning with basic assumptions and a proposed grading plan for the site. The hydraulic features of the channels were initially assumed and then confirmed through multiple iterations.

The AutoCAD Civil 3D software package was used to generate the proposed site grading plan and subsequently to determine drainage areas, volumes, and other site geometry. HydroCAD (version 10.00) modeling software was used to conduct the hydrologic calculations. The Federal Highway Administration's Hydraulic Toolbox was used to conduct hydraulic calculations for this analysis with inputs based on the site geometry, rainfall data, and other design assumptions.

The HydroCAD model was used to generate peak flow rates for the upstream watershed features. Hydraulic Toolbox was used to generate channel velocities and water surface elevations at the most critical points along the channel alignment. The proposed channels will pass flows equal to and lesser than those generated by the 24-hour, 25-year storm.

V. NOTES/ASSUMPTIONS

The following is a list of key notes and assumptions made in completing this analysis.

- A design parameter minimum six (6) inches of freeboard was used.
- Within the HydroCAD program, the runoff was calculated using the SCS TR-20 method.
- Runoff curve numbers (CN) used in the analysis were as follows:
  - 78 for landfill vegetated final cap cover and offsite areas,
  - 98 for sediment basin water surface.
  - 77 for exposed and disturbed soil (daily cover); however exposed and disturbed soil was ignored to be conservative and 78 was used within landfill construction footprint.
- The time of concentration was calculated using the Curve Number Method in HydroCAD which takes inputs for each drainage area of the longest hydraulic flow path and average land slope.

VI. SUMMARY OF RESULTS

The channels properly control and convey the water volume generated by the 24-hour, 25-year storm such that the peak water surface elevation stays safely below the impounding sideslopes with greater than six (6) inches of freeboard. A turf reinforced matting (TRM) channel lining was chosen for all channels as the maximum peak design velocity is under 11 fps. Appendix C includes specifications for

GreenArmor TRM with permissible vegetated velocities of 16 fps exceeding the maximum design peak velocity of 11 fps. Riprap maybe used as a replacement to TRM where deemed necessary.

Table 1 – Perimeter Ditch Summary

Parameter	Peak Discharge (cfs)	Slope (ft/ft)	Flow Depth (feet)	Freeboard (feet)	Velocity (fps)	Shear Stress (lb/ft^2)
Max. Depth	83.65	0.02	1.709	1.291	6.025	1.255
Max. Velocity	83.65	0.08	1.237	1.763	10.071	3.830

VII. CONCLUSIONS

The proposed grading of the landfill site in combination with the design of the channels as presented above is sufficient to safely control and convey the 24-hour, 25-year storm as stipulated by TDEC regulations. Calculations for proper conveyance of upstream and downstream flows were not presented in this analysis and will be performed separately. Refer to accompanying calculations for upstream and downstream conveyance design features (storm water terraces and letdowns, culverts and sediment basins) conveyance features.

VIII. ATTACHMENTS

Figures:
Exhibit 1: Drainage Map

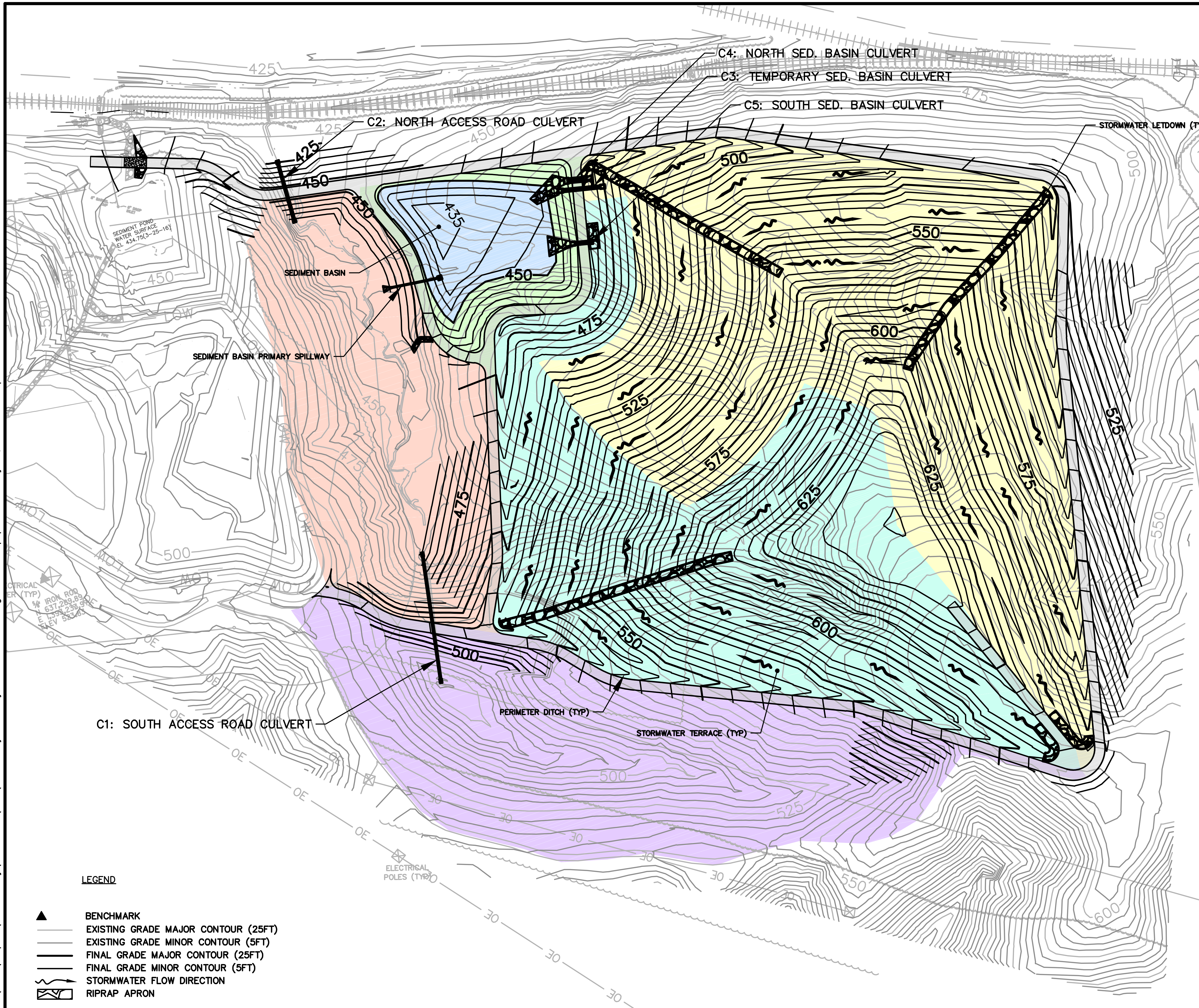
Attachments:
Attachment A: NOAA Precipitation Frequency Data
Attachment B: Perimeter Ditch: Drainage Map, HydroCAD report, and Hydraulic Toolbox Report
Attachment C: Turf Reinforced Matting: GreenArmor System

IX. REFERENCES

1. TDEC, *"Rules of Tennessee Department of Environment and Conservation, Chapter 0400-11-01 – Solid Waste Processing and Disposal"*, Solid Waste Management, September 2012.

EXHIBIT 1

Drainage Map



LEGEND

- BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (25FT)
- EXISTING GRADE MINOR CONTOUR (5FT)
- FINAL GRADE MAJOR CONTOUR (25FT)
- FINAL GRADE MINOR CONTOUR (5FT)
- STORMWATER FLOW DIRECTION
- RIPRAP APRON

DRAINAGE AREAS

NORTH SED. BASIN CULVERT AREA = 15.99 ACRES CN = 78 TC = 14.0 MINS
SOUTH SED. BASIN CULVERT AREA = 12.06 ACRES CN = 78 TC = 16.5 MINS
SED. BASIN OPEN WATER AREA = 1.50 ACRES CN = 98 TC = 5.0 MINS
SED. BASIN SOIL AREA = 1.50 ACRES CN = 78 TC = 5.0 MINS
NORTH ACCESS ROAD CULVERT AREA = 7.260 CN = 49 TC = 6.0 MINS
SOUTH ACCESS ROAD CULVERT AREA-1 = 9.24 ACRES; CN-1 = 49 AREA-2 = 1 ACRE; CN-2 = 77 TC = 9.9 MINS

AECOM

SCEPTER, INC.

WAVERLY, TENNESSEE

ROAD CULVERT DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/4/17	1A

ATTACHMENT A

NOAA Precipitation Frequency Data

NOAA Atlas 14, Volume 2, Version 3
Location name: Waverly, Tennessee, US*
Latitude: 36.0689°, Longitude: -87.9462°
Elevation: 523 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.405 (0.368-0.450)	0.476 (0.434-0.529)	0.547 (0.497-0.607)	0.604 (0.548-0.668)	0.675 (0.610-0.746)	0.728 (0.655-0.803)	0.781 (0.699-0.862)	0.832 (0.742-0.917)	0.898 (0.794-0.990)	0.948 (0.833-1.05)
10-min	0.647 (0.588-0.718)	0.761 (0.694-0.845)	0.876 (0.796-0.972)	0.965 (0.877-1.07)	1.08 (0.972-1.19)	1.16 (1.04-1.28)	1.24 (1.11-1.37)	1.32 (1.18-1.45)	1.42 (1.26-1.57)	1.49 (1.31-1.65)
15-min	0.809 (0.735-0.898)	0.957 (0.872-1.06)	1.11 (1.01-1.23)	1.22 (1.11-1.35)	1.36 (1.23-1.51)	1.47 (1.32-1.62)	1.57 (1.40-1.73)	1.66 (1.48-1.83)	1.79 (1.58-1.97)	1.88 (1.65-2.07)
30-min	1.11 (1.01-1.23)	1.32 (1.20-1.47)	1.58 (1.43-1.75)	1.77 (1.61-1.96)	2.02 (1.83-2.23)	2.21 (1.99-2.44)	2.40 (2.15-2.65)	2.59 (2.31-2.86)	2.84 (2.51-3.14)	3.04 (2.67-3.35)
60-min	1.38 (1.26-1.54)	1.66 (1.51-1.84)	2.02 (1.83-2.24)	2.30 (2.09-2.55)	2.69 (2.43-2.97)	2.99 (2.70-3.31)	3.31 (2.96-3.65)	3.63 (3.24-4.01)	4.08 (3.61-4.50)	4.43 (3.89-4.89)
2-hr	1.59 (1.44-1.76)	1.91 (1.73-2.11)	2.33 (2.11-2.59)	2.69 (2.43-2.98)	3.19 (2.87-3.52)	3.60 (3.22-3.98)	4.03 (3.59-4.46)	4.49 (3.98-4.97)	5.14 (4.52-5.70)	5.67 (4.95-6.29)
3-hr	1.72 (1.57-1.89)	2.05 (1.87-2.27)	2.52 (2.29-2.78)	2.91 (2.64-3.20)	3.45 (3.12-3.80)	3.91 (3.52-4.30)	4.39 (3.93-4.82)	4.90 (4.37-5.39)	5.64 (4.98-6.20)	6.24 (5.47-6.88)
6-hr	2.13 (1.93-2.35)	2.53 (2.30-2.80)	3.10 (2.82-3.43)	3.58 (3.25-3.95)	4.26 (3.84-4.69)	4.83 (4.33-5.32)	5.43 (4.85-5.98)	6.09 (5.40-6.70)	7.01 (6.17-7.72)	7.78 (6.78-8.58)
12-hr	2.58 (2.36-2.85)	3.08 (2.82-3.40)	3.79 (3.46-4.18)	4.38 (3.99-4.82)	5.22 (4.73-5.74)	5.92 (5.34-6.51)	6.66 (5.97-7.32)	7.46 (6.65-8.20)	8.60 (7.59-9.45)	9.54 (8.36-10.5)
24-hr	3.14 (2.88-3.44)	3.76 (3.45-4.12)	4.64 (4.25-5.09)	5.37 (4.91-5.87)	6.39 (5.82-6.98)	7.23 (6.56-7.89)	8.12 (7.32-8.85)	9.06 (8.12-9.88)	10.4 (9.22-11.3)	11.5 (10.1-12.5)
2-day	3.75 (3.46-4.10)	4.49 (4.14-4.92)	5.53 (5.09-6.04)	6.36 (5.84-6.93)	7.52 (6.89-8.19)	8.46 (7.72-9.21)	9.45 (8.56-10.3)	10.5 (9.44-11.4)	11.9 (10.6-13.0)	13.1 (11.6-14.3)
3-day	4.01 (3.69-4.37)	4.80 (4.42-5.24)	5.89 (5.43-6.43)	6.77 (6.22-7.37)	7.98 (7.31-8.69)	8.96 (8.17-9.75)	9.97 (9.04-10.9)	11.0 (9.94-12.0)	12.5 (11.2-13.7)	13.6 (12.1-15.0)
4-day	4.26 (3.93-4.64)	5.11 (4.71-5.56)	6.26 (5.77-6.82)	7.18 (6.61-7.81)	8.45 (7.74-9.18)	9.46 (8.62-10.3)	10.5 (9.52-11.4)	11.6 (10.4-12.6)	13.0 (11.7-14.3)	14.2 (12.6-15.6)
7-day	5.06 (4.65-5.50)	6.06 (5.58-6.59)	7.43 (6.83-8.07)	8.52 (7.81-9.25)	10.0 (9.17-10.9)	11.3 (10.2-12.2)	12.6 (11.4-13.6)	13.9 (12.5-15.1)	15.7 (14.0-17.2)	17.2 (15.2-18.8)
10-day	5.77 (5.34-6.22)	6.90 (6.38-7.45)	8.39 (7.75-9.05)	9.55 (8.81-10.3)	11.1 (10.2-12.0)	12.4 (11.3-13.3)	13.6 (12.4-14.7)	14.9 (13.5-16.1)	16.6 (15.0-18.0)	18.0 (16.1-19.6)
20-day	7.88 (7.35-8.46)	9.36 (8.74-10.1)	11.1 (10.4-12.0)	12.5 (11.6-13.4)	14.2 (13.2-15.3)	15.5 (14.4-16.7)	16.8 (15.5-18.1)	18.1 (16.6-19.5)	19.7 (18.0-21.2)	20.9 (19.0-22.6)
30-day	9.63 (9.00-10.3)	11.4 (10.7-12.2)	13.4 (12.6-14.4)	15.0 (14.0-16.0)	16.9 (15.8-18.1)	18.4 (17.1-19.7)	19.9 (18.5-21.3)	21.4 (19.7-22.9)	23.2 (21.3-24.9)	24.6 (22.5-26.4)
45-day	12.1 (11.3-12.9)	14.3 (13.4-15.2)	16.7 (15.7-17.8)	18.5 (17.3-19.7)	20.8 (19.4-22.1)	22.4 (20.9-23.9)	24.1 (22.4-25.6)	25.6 (23.7-27.3)	27.6 (25.4-29.5)	29.0 (26.6-31.0)
60-day	14.5 (13.6-15.4)	17.1 (16.0-18.2)	19.8 (18.6-21.1)	21.8 (20.4-23.2)	24.3 (22.7-25.9)	26.1 (24.4-27.9)	27.8 (25.9-29.7)	29.5 (27.4-31.5)	31.4 (29.1-33.6)	32.9 (30.3-35.2)

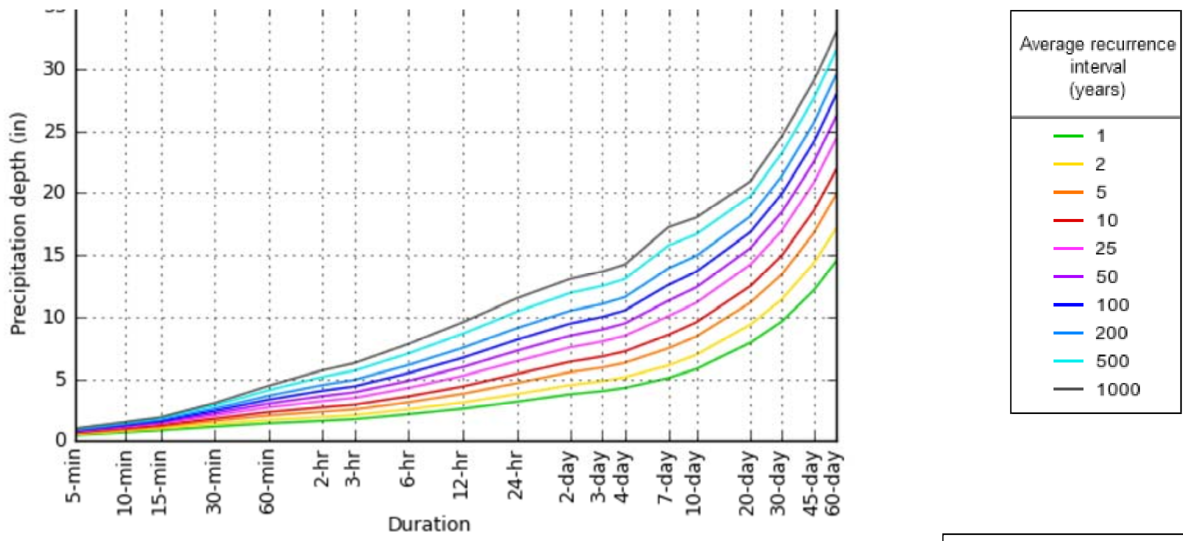
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

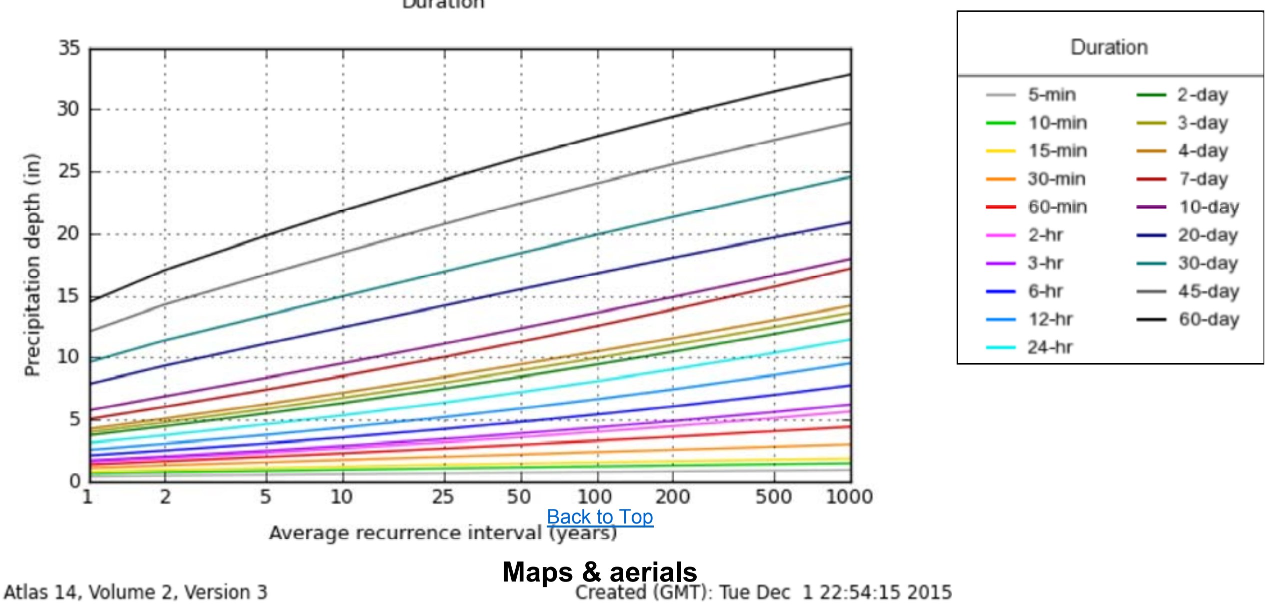
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical



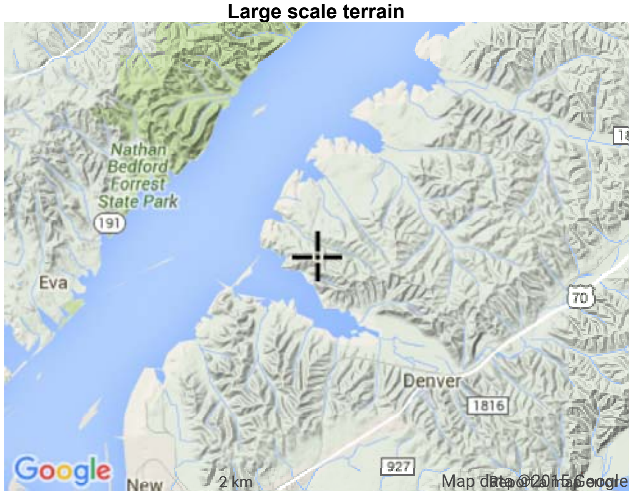


Back to Top

Maps & aerials

Small scale terrain

Large scale terrain



Large scale map

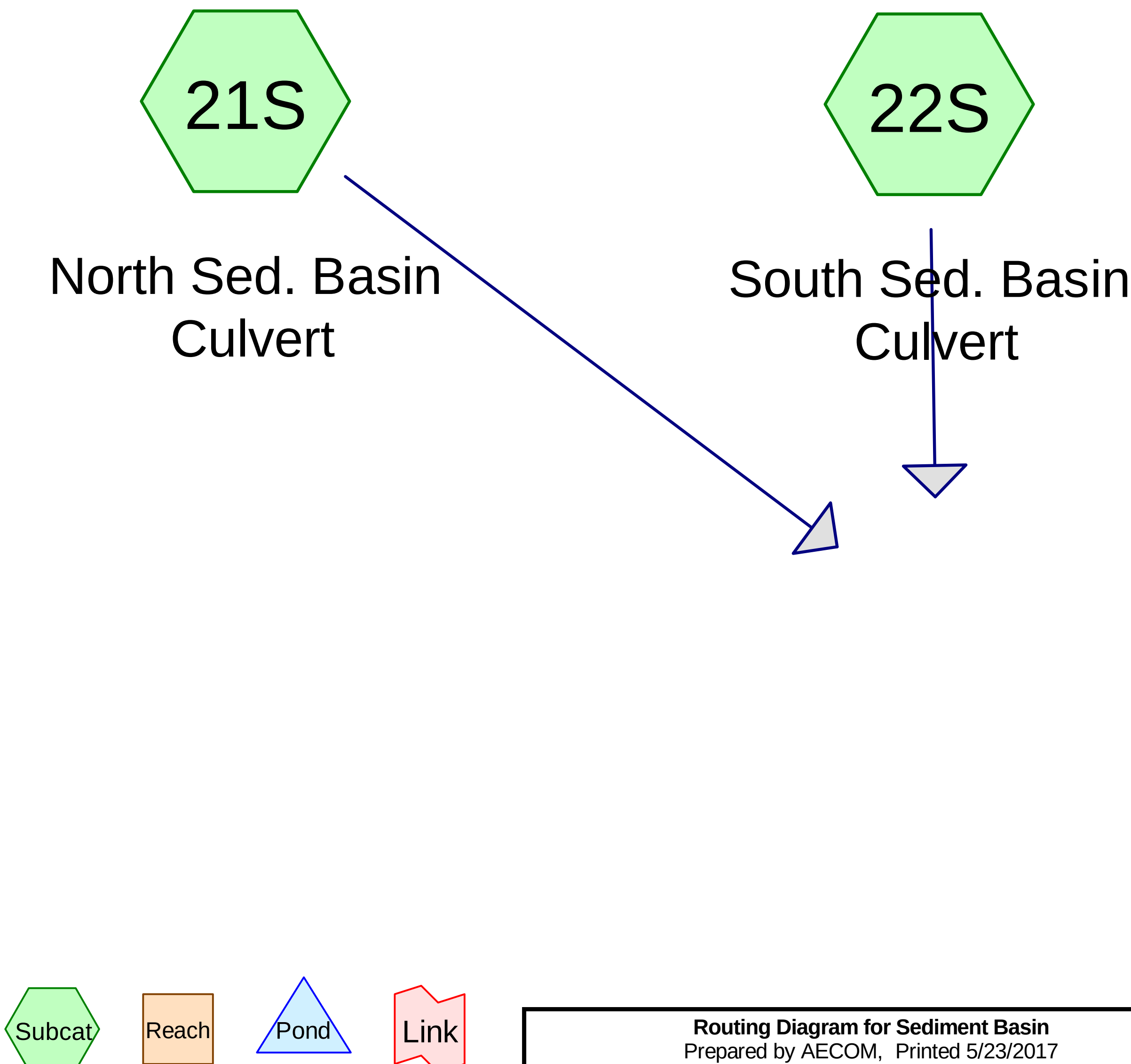
Large scale aerial

[Back to Top]

US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910

ATTACHMENT B

Perimeter Ditch Design



Routing Diagram for Sediment Basin
Prepared by AECOM, Printed 5/23/2017
HydroCAD® 10.00 s/n 01089 © 2013 HydroCAD Software Solutions LLC

Summary for Subcatchment 21S: North Sed. Basin Culvert

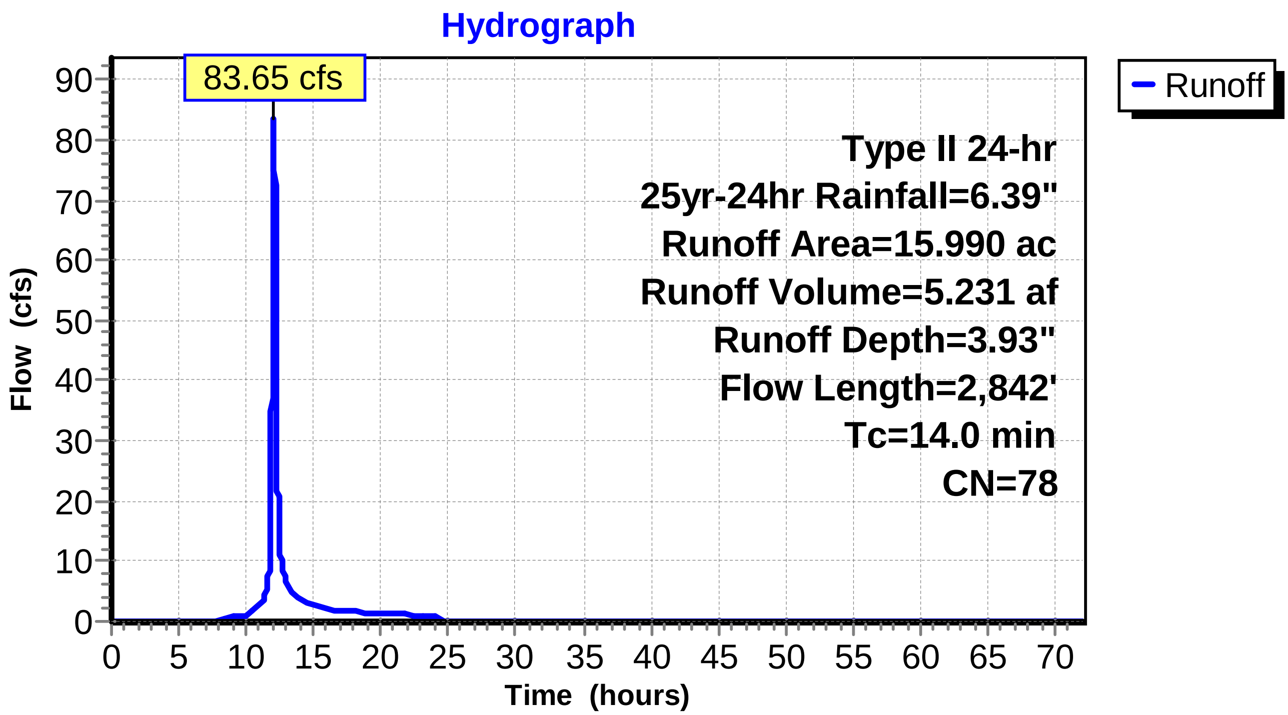
Runoff = 83.65 cfs @ 12.06 hrs, Volume= 5.231 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	15.990	78	
	15.990		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3333	4.04		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	116	0.1042	17.75	496.95	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
0.9	916	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
1.1	592	0.0250	9.32	335.59	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.6	554	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 3** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.1	55	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 4** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
14.0	2,842	Total			

Subcatchment 21S: North Sed. Basin Culvert



Hydrograph for Subcatchment 21S: North Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.06
8.00	0.77	0.01	0.25
9.00	0.94	0.04	0.62
10.00	1.16	0.10	1.12
11.00	1.50	0.23	2.67
12.00	4.24	2.08	**73.26**
13.00	4.93	2.65	**6.29**
14.00	5.24	2.92	3.64
15.00	5.45	3.10	2.81
16.00	5.62	3.25	2.20
17.00	5.76	3.37	1.90
18.00	5.89	3.48	1.68
19.00	5.99	3.57	1.46
20.00	6.08	3.65	1.23
21.00	6.16	3.72	1.15
22.00	6.24	3.79	1.11
23.00	6.32	3.86	1.07
24.00	**6.39**	**3.93**	1.02
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 22S: South Sed. Basin Culvert

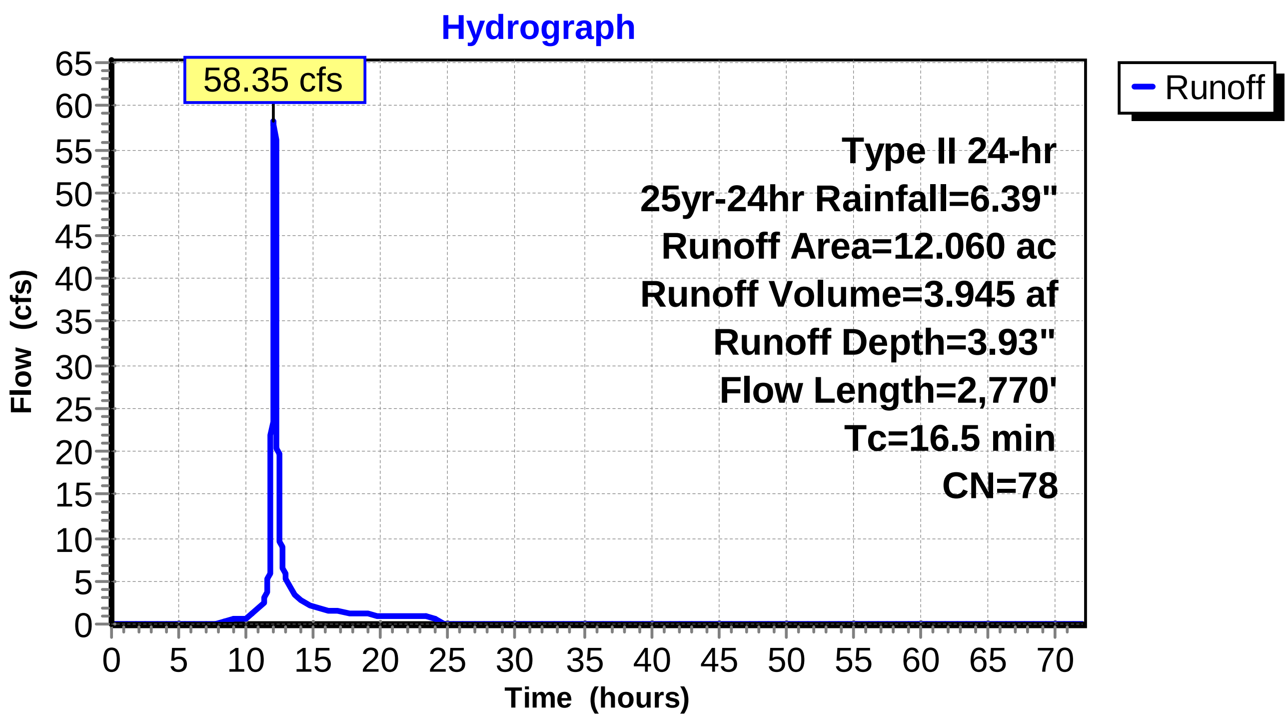
Runoff = 58.35 cfs @ 12.08 hrs, Volume= 3.945 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	12.060	78	
	12.060		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Let Down** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
1.7	1,668	0.0800	16.68	600.32	**Channel Flow, Perimeter Ditch 1** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
0.9	429	0.0200	8.34	300.16	**Channel Flow, Perimeter Ditch 2** Area= 36.0 sf Perim= 22.0' r= 1.64' n= 0.035
16.5	2,770	Total			

Subcatchment 22S: South Sed. Basin Culvert



Hydrograph for Subcatchment 22S: South Sed. Basin Culvert

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.04
8.00	0.77	0.01	0.18
9.00	0.94	0.04	0.45
10.00	1.16	0.10	0.82
11.00	1.50	0.23	1.94
12.00	4.24	2.08	**46.36**
13.00	4.93	2.65	**4.91**
14.00	5.24	2.92	2.80
15.00	5.45	3.10	2.13
16.00	5.62	3.25	1.68
17.00	5.76	3.37	1.44
18.00	5.89	3.48	1.27
19.00	5.99	3.57	1.11
20.00	6.08	3.65	0.94
21.00	6.16	3.72	0.87
22.00	6.24	3.79	0.84
23.00	6.32	3.86	0.81
24.00	**6.39**	**3.93**	0.77
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Hydraulic Analysis Report

Project Data

Project Title:

Designer:

Project Date: Thursday, March 24, 2016

Project Units: U.S. Customary Units

Notes:

Channel Analysis: Run-off Ditch (Max Depth)

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 3.0000 ft/ft

Side Slope 2 (Z2): 3.0000 ft/ft

Channel Width: 3.0000 ft

Longitudinal Slope: 0.0200 ft/ft

Manning's n: 0.0350

Flow: 83.6500 cfs

Result Parameters

Depth: 1.7086 ft

Area of Flow: 13.8839 ft^2

Wetted Perimeter: 13.8062 ft

Hydraulic Radius: 1.0056 ft

Average Velocity: 6.0250 ft/s

Top Width: 13.2516 ft

Froude Number: 1.0373

Critical Depth: 1.7393 ft

Critical Velocity: 5.8525 ft/s

Critical Slope: 0.0185 ft/ft

Critical Top Width: 13.44 ft

Calculated Max Shear Stress: 2.1323 lb/ft^2

Calculated Avg Shear Stress: 1.2550 lb/ft^2

Channel Analysis: Run-off Ditch (Max V)

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 3.0000 ft/ft

Side Slope 2 (Z2): 3.0000 ft/ft

Channel Width: 3.0000 ft

Longitudinal Slope: 0.0800 ft/ft

Manning's n: 0.0350

Flow: 83.6500 cfs

Result Parameters

Depth: 1.2375 ft

Area of Flow: 8.3063 ft^2

Wetted Perimeter: 10.8264 ft

Hydraulic Radius: 0.7672 ft

Average Velocity: 10.0706 ft/s

Top Width: 10.4248 ft

Froude Number: 1.9882

Critical Depth: 1.7387 ft

Critical Velocity: 5.8554 ft/s

Critical Slope: 0.0185 ft/ft

Critical Top Width: 13.43 ft

Calculated Max Shear Stress: 6.1774 lb/ft^2

Calculated Avg Shear Stress: 3.8300 lb/ft^2

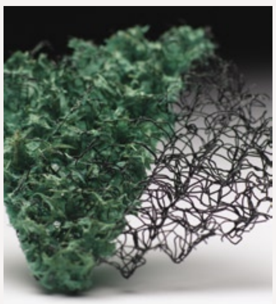
ATTACHMENT C

Turf Reinforced Matting: GreenArmor System

GREENARMOR™ SYSTEM

GreenArmor™ 7010

Extend the Boundaries of Natural Vegetation



The GreenArmor™ System begins with Enkamat® TRM (Turf Reinforcement Mat), which provides a permanent, lofty and open matrix. It is then hydraulically infilled with Flexterra® FGM™ (Flexible Growth Medium™) to intimately bond soil and seeds while accelerating growth. This unique system protects against elevated levels of hydraulic lift and shear forces while encouraging turf establishment and long-term root reinforcement. The synergistic combination of cost effective technologies enables the GreenArmor System to provide unprecedented levels of design safety.

Enkamat 7010

Enkamat 7020

GENERAL

1.01 SUMMARY

(Section 35 43 00 - Permanent Geosynthetic Turf Reinforcement Mat)

A. This section specifies a permanent Geosynthetic Turf Reinforcement Mat (TRM) with Flexible Growth Medium (FGM) infill, to prevent long-term soil and vegetation loss resulting from excessive water flow (velocity and shear stress) in which unreinforced vegetation could not resist. The FGM provides immediate and temporary protection against movement and/or loss of soil until vegetation can be established. The FGM infill also provides an ideal environment for rapid seed germination and accelerated plant and root establishment within the matrix of the TRM.

B. Related Sections: Other Specification Sections, which directly relate to the work of this Section include, but are not limited to the following:

1. *Section 01 57 13 - Temporary Erosion and Sedimentation Controls*
2. *Section 31 25 00 - Erosion and Sedimentation Controls*
3. *Section 31 35 00 - Slope Protection*
4. *Section 32 91 00 - Planting Preparation*
5. *Section 32 92 00 - Turf and Grasses*
6. *Section 35 30 00 - Shoreline Protection*

1.02 SUBMITTALS

A. Product Data: Submit manufacturer's product data and installation instructions. Include required substrate preparation, list of materials and application rate.

B. Certifications: Manufacturer shall submit a letter of certification that the product meets or exceeds all physical property, endurance, performance and packaging requirements.

1.03 DELIVERY, STORAGE AND HANDLING

A. Deliver materials and products in UV and weather-resistant factory labeled packages. Store and handle in strict compliance with manufacturer's instructions and recommendations. Protect from damage, weather, excessive temperatures and construction operations.

PRODUCTS

2.01 MANUFACTURERS/OR THEIR REPRESENTATIVES

A. PROFILE Products LLC
750 Lake Cook Road—Suite 440
Buffalo Grove, IL 60089
800-508-8681 (Fax 847-215-0577)
www.profileproducts.com

2.02 MATERIALS

A. Turf Reinforcement Mat shall be Enkamat 7010, manufactured for the purpose of permanent channel lining and turf reinforcement. The TRM shall be made from 100% synthetic material and contain no biodegradable or photodegradable components or materials.

1. The TRM shall be a homogeneous, three-dimensional matrix made of continuous monofilament yarns which are thermally fused at the crossover points to provide a structure that will maintain its three dimensional stability without laminated or stitched layers. No nettings or stitching shall be permitted. The TRM shall have a sufficient Area Holding Capacity and a minimum 95% open space available for soil, FGM and root interaction. The TRM shall not lose its structural integrity and shall not unravel or separate when TRM is cut in the field.
2. The TRM shall exhibit no buoyancy factor (i.e., the specific gravity of the fibers used should be greater than 1.0) so as to allow the TRM to maintain intimate contact with the soil (particularly between fasteners) under low flow or submersed conditions.

PROFILE Products LLC
750 Lake Cook Rd. • Suite 440
Buffalo Grove, IL 60089 • 800-508-8681
www.profileproducts.com

3. The TRM, when infilled with FGM, shall meet the property values noted.

B. Flexible Growth Medium for hydraulic infill of TRM shall be Flexterra® FGM™ and conform to the property values as presented in the Flexterra data sheet.

All components of the FGM shall be pre-packaged by the Manufacturer to assure material performance and in compliance with the following values. **Under no circumstances will field mixing of additives or components be accepted.**

Thermally Processed Wood Fibers – 74.5% ± 3.5%

Proprietary Crosslinked Hydro-Colloid Tackifiers and Activators – 10% ± 1%

Proprietary Crimped, Interlocking Fibers – 5% ±1%

Moisture Content – 10.5% ± 1.5%

	TEST METHOD	ENGLISH	SI
PHYSICAL			
Mass Per Unit Area	ASTM D6566	19.5 oz/yd²	661 g/m²
Thickness	ASTM D6525	0.40 in	10 mm
Tensile Strength - MD	ASTM D6818	170 lb/ft	2.5 kN/m²
Light Penetration	ASTM D6567	1.0%	1.0%
Ground Cover	ASTM D6567	99.0%	99.0%
Absorption	ASTM D1117	498.0%	498.0%
UV Resistance	ASTM D7238 & D6818	80.0%	80.0%
Bench Scale C-Factor[2] (average)	ASTM D7101	0.01	0.01
Resiliency	ASTM D6524	90.0%	90.0%
ENDURANCE			
Functional Longevity[1]	Observed	>36 months	>36 months
PERFORMANCE			
C-Factor	Large Scale[3]	0.01	0.01
Manning's n Range	ASTM D6460	0.022 - 0.045	0.022 - 0.045
Permissible Vegetated Shear	ASTM D6460	8.0 lb/ft²	0.38 kN/m²
Permissible Vegetated Velocity	ASTM D6460	16.0 ft/s	4.9 m/s
Permissible Unvegetated Shear	ASTM D6460	3.3 lb/ft²	0.16 kN/m²
Permissible Unvegetated Velocity	ASTM D6460	12.0 ft/s	3.7 m/s
Vegetation Establishment	ASTM D7322	800%	800%

1. Functional longevity depends on moisture, light and environmental conditions.
2. Cover Factor is calculated as soil loss ratio of treated surface versus an untreated control surface.
3. Large scale testing conducted at Utah Water Research facility using rainfall simulator on 2.5H:1V slope, sandy-loam soil, at a rate of 5" per hour for a duration of 60 minutes.

EXECUTION

3.01 PREPARATION

A. The installation site shall be prepared by clearing, grubbing and excavation or filling the area to the design grade.

B. The surface to receive the TRM shall be prepared to relatively smooth conditions free of obstructions, rocks, dirt clods, roots, stumps, depressions, debris and soft or low density pockets of material. The material shall be capable of supporting a vegetative cover.

C. Erosion features such as rills, gullies, etc. must be graded out of the surface before TRM deployment. Smooth roll drum compaction will be required before deploying TRM to make sure the TRM makes immediate contact with the soil and to ensure that the soil has been compacted.

D. Cut trenches for initial anchor trenches, termination trench and longitudinal anchor trenches (12 inches wide and 12 inches in depth) as shown on the drawings.

3.02 INSTALLATION

A. Care shall be taken during installation to avoid damage occurring to the TRM as a result of the installation process. Should the TRM be damaged during installation, a TRM patch shall be placed over the damaged area extending 1 m (3.28 ft) beyond the perimeter of the damage.

B. Install anchoring devices at a frequency of 2½ pins/staples per square yard. Additional anchoring devices may be required depending on site conditions or alignment of the slope or channel. Always staple (1' centers) the seams between individual TRM rolls.

C. When overlapping successive TRM rolls, the rolls shall be overlapped upstream over downstream and/or upslope over downslope.

D. For channels, begin at the downstream end in the center of the channel. Inspect trenches for position accuracy and depth and re-dig to required dimensions. If trenches have not yet been constructed, dig initial anchor trenches, check slot trenches and longitudinal anchor trenches as illustrated in installation guidelines or as directed on the plans. Unroll approximately 10' of the TRM, positioning the roll face down (as it unrolls) over the initial anchor trench, extending several inches beyond the trench with the roll sitting on the down stream side of the anchor trench. Positioning roll in this manner permits backfilling and compaction of soil into the trench while allowing installer to proceed with proper deployment of TRM by unrolling upstream, over the anchor trench.

E. Position second TRM with a minimum 4-inch overlap of the previous TRM and secure it into the anchor trench. After entire width area is installed with the TRM, then backfill and compact the anchor trench.

F. Continue deploying TRM upstream to the next check slot. Overlay a minimum of 18 inches the ends of rolls with the next roll(s) being deployed, or position in bottom of check slot, anchor and backfill and compact check slots. Continue the processes until you reach the upstream starting point of the TRM.

G. For slopes, construct top anchor trench 1-3' beyond crest of slope, or as illustrated in drawings or shown in manufacturers recommended installation guidelines. Position TRM roll at crest of slope with sufficient material to line the entire anchor trench plus enough material left over to cover the trench. Position adjacent rolls to facilitate 6" overlaps. Anchor TRM in trench with appropriate pins/staples at 1' centers. Once several rolls are anchored in trench, begin to backfill and compact trench to original elevation. The preferred method of deploying roll down slope is to stand in front of the roll and pin it as it rolls out down the slope, minimizing foot traffic on TRM, which will eliminate depressions under the mat. Always allow the mat to drape over the soil, never pulling it taut, to minimize tenting. Place additional pins into any apparent depressions to maintain contact with the soil.

H. Hydraulically fill the TRM with 0.35 inches of FGM, applied with hose at close range. Optimum application rate is 3500 lbs/acre or to the depth of where the tips of TRM are still exposed.

I. Strictly comply with FGM manufacturer's installation instructions and recommendations. For optimum FGM pumping and application performance, use approved mechanically agitated, hydraulic seeding/mulching machines, hose of sufficient length to reach the TRM, use of a 50 degree tip/nozzle is highly recommended. Apply FGM from hose positioned over shoulder with nozzle approximately at chest level (48-60") to achieve optimum TRM infill.

J. For optimum hydraulic performance and vegetative establishment, be careful not to overfill the TRM. The tips of the TRM shall be slightly exposed.

K. Apply supplemental water over the area as directed by site personnel during germination and initial three months of vegetation growth.

3.03 CLEANING AND PROTECTION

A. Clean spills promptly. Advise owner of methods for protection of treated areas. Do not allow treated areas to be trafficked or subjected to grazing.

An electronic text file of this CSI formatted specification can be obtained by contacting a technical service representative at 800-508-8681.

APPENDIX F.1D

Letdown Design (Rev 3)

I. PURPOSE

The purpose of this analysis is to design the proposed landfill cap letdown channels for the Scepter Class II Disposal Facility in Waverly, TN consistent with state regulations.

II. SITE AND PROJECT DESCRIPTION

The letdown design was performed as part of the Part II Solid Waste Permit Application. The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management. Refer to Exhibit 1 for a depiction of the letdown locations in relation to the landfill.

The following sections summarize the design criteria, procedure, assumptions, and results of the letdown design.

III. REGULATORY REQUIREMENTS / DESIGN CRITERIA

The below parts of the TDEC regulations specify requirements for the design of the letdowns:

Rule 0400-11-01-.04(2)(i)2.

The operator must design, construct, operate, and maintain a run-off management system to collect and control at least the peak flow resulting from a 24-hour, 25 year storm.

The results of the analysis presented herein show that the letdowns are designed to collect and control at least the peak flow resulting from a 25-year/24-hour storm. The letdown channels will convey landfill run-off flow towards the perimeter channels.

Rule 0400-11-01-.04(2)(i)6.

The operator must take other erosion control measure (e.g., temporary mulching or seeding, silt barriers) as necessary to control erosion of the site.

Proper channel lining will be selected based on anticipated velocities such that erosion within the letdowns will be significantly reduced.

IV. PROCEDURE

Design of the landfill site stormwater features was an iterative process beginning with basic assumptions and a proposed grading plan for the site. The hydraulic features of the letdowns were initially assumed and then confirmed through multiple iterations.

The AutoCAD Civil 3D software package was used to generate the proposed site grading plan and subsequently to determine drainage areas, volumes, and other site geometry. HydroCAD (version 10.00) modeling software was used to conduct the hydrologic calculations. The Federal Highway Administration's Hydraulic Toolbox was used to conduct hydraulic calculations for this analysis with inputs based on the site geometry, rainfall data, and other design assumptions.

The HydroCAD model was used to generate peak flow rates for the upstream watershed features. Hydraulic Toolbox was used to generate channel velocities and water surface elevations at the most critical points along the letdown alignment. The proposed channels will pass flows equal to and lesser than those generated by the 24-hour, 25-year storm.

V. NOTES/ASSUMPTIONS

The following is a list of key notes and assumptions made in completing this analysis.

- A design parameter minimum six (6) inches of freeboard was used.
- Within the HydroCAD program, the runoff was calculated using the SCS TR-20 method.
- Runoff curve numbers (CN) used in the analysis were as follows:
    - 78 for landfill vegetated final cap cover and offsite areas,
    - 98 for sediment basin water surface.
    - 77 for exposed and disturbed soil (daily cover); however exposed and disturbed soil was ignored to be conservative and 78 was used within landfill construction footprint.
- The time of concentration was calculated using the Curve Number Method in HydroCAD which takes inputs for each drainage area of the longest hydraulic flow path and average land slope.

VI. SUMMARY OF RESULTS

The letdowns properly control and convey the water volume generated by the 24-hour, 25-year storm such that the average flow depth stays safely below the impounding sideslopes with greater than six inches of freeboard. A channel lining of riprap was chosen for all letdowns as the permissible shear stress ranges from 1.47 to 4.35 lb/ft^2 and the peak velocity ranged from 5.33 to 9.35 fps. See Table 1 for further detail.

Table 1 - Letdown Summary

Letdown ID	Peak Discharge (cfs)	Slope (ft/ft)	Flow Depth (feet)	Freeboard (feet)	Velocity (fps)	Shear Stress (lb/ft^2)
Northwest	36.21	0.2090	0.361	1.639	9.348	4.349

Northeast	32.06	0.1853	0.349	1.651	8.599	3.730
Southeast A	18.17	0.0791	0.320	1.680	5.334	1.471
Southeast B	19.87	0.1042	0.311	1.689	6.008	1.888
Southwest	36.99	0.1866	0.379	1.621	9.076	4.058

VII. CONCLUSIONS

The proposed grading of the landfill site in combination with the design of the letdowns as presented above is sufficient to safely control and convey the 24-hour, 25-year storm as stipulated by TDEC regulations. Calculations for proper conveyance of upstream and downstream flows were not presented in this analysis and are performed separately. Refer to accompanying calculations for upstream and downstream conveyance design features (storm water terraces, ditches, culverts and sediment basins).

VIII. ATTACHMENTS

Figures:

Exhibit 1: Drainage Map

Attachments:

Attachment A: NOAA Precipitation Frequency Data
Attachment B: Letdown: Drainage Map, HydroCAD report, and Hydraulic Toolbox Report

IX. REFERENCES

1. TDEC, *"Rules of Tennessee Department of Environment and Conservation, Chapter 0400-11-01 – Solid Waste Processing and Disposal"*, Solid Waste Management, September 2012.

EXHIBIT 1

Drainage Map

DRAINAGE AREAS

NORTHWEST LETDOWN	AREA = 5.93 ACRES CN = 78 TC = 5.0 MINS
NORTHEAST LETDOWN	AREA = 5.25 ACRES CN = 78 TC = 5.0 MINS
SOUTHEAST LETDOWN A	AREA = 3.46 ACRES CN = 78 TC = 5.0 MINS
SOUTHEAST LETDOWN B	AREA = 3.46 ACRES CN = 78 TC = 5.0 MINS
SOUTHWEST LETDOWN	AREA = 7.01 ACRES CN = 78 TC = 5.0 MINS

LEGEND

- ▲ BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (25FT)
- EXISTING GRADE MINOR CONTOUR (5FT)
- FINAL GRADE MAJOR CONTOUR (25FT)
- FINAL GRADE MINOR CONTOUR (5FT)
- STORMWATER FLOW DIRECTION

AECOM

SCEPTER, INC.
WAVERLY, TENNESSEE

LETDOWN DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/9/17	1

S:\2016\Scepter\028 Waverly\02 Landfill\CAD\x-tributary boundary-letdown.dwg User:nick.popkowski May 23, 2017 - 8:22am

ATTACHMENT A

NOAA Precipitation Frequency Data

NOAA Atlas 14, Volume 2, Version 3
Location name: Waverly, Tennessee, US*
Latitude: 36.0689°, Longitude: -87.9462°
Elevation: 523 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.405 (0.368-0.450)	0.476 (0.434-0.529)	0.547 (0.497-0.607)	0.604 (0.548-0.668)	0.675 (0.610-0.746)	0.728 (0.655-0.803)	0.781 (0.699-0.862)	0.832 (0.742-0.917)	0.898 (0.794-0.990)	0.948 (0.833-1.05)
10-min	0.647 (0.588-0.718)	0.761 (0.694-0.845)	0.876 (0.796-0.972)	0.965 (0.877-1.07)	1.08 (0.972-1.19)	1.16 (1.04-1.28)	1.24 (1.11-1.37)	1.32 (1.18-1.45)	1.42 (1.26-1.57)	1.49 (1.31-1.65)
15-min	0.809 (0.735-0.898)	0.957 (0.872-1.06)	1.11 (1.01-1.23)	1.22 (1.11-1.35)	1.36 (1.23-1.51)	1.47 (1.32-1.62)	1.57 (1.40-1.73)	1.66 (1.48-1.83)	1.79 (1.58-1.97)	1.88 (1.65-2.07)
30-min	1.11 (1.01-1.23)	1.32 (1.20-1.47)	1.58 (1.43-1.75)	1.77 (1.61-1.96)	2.02 (1.83-2.23)	2.21 (1.99-2.44)	2.40 (2.15-2.65)	2.59 (2.31-2.86)	2.84 (2.51-3.14)	3.04 (2.67-3.35)
60-min	1.38 (1.26-1.54)	1.66 (1.51-1.84)	2.02 (1.83-2.24)	2.30 (2.09-2.55)	2.69 (2.43-2.97)	2.99 (2.70-3.31)	3.31 (2.96-3.65)	3.63 (3.24-4.01)	4.08 (3.61-4.50)	4.43 (3.89-4.89)
2-hr	1.59 (1.44-1.76)	1.91 (1.73-2.11)	2.33 (2.11-2.59)	2.69 (2.43-2.98)	3.19 (2.87-3.52)	3.60 (3.22-3.98)	4.03 (3.59-4.46)	4.49 (3.98-4.97)	5.14 (4.52-5.70)	5.67 (4.95-6.29)
3-hr	1.72 (1.57-1.89)	2.05 (1.87-2.27)	2.52 (2.29-2.78)	2.91 (2.64-3.20)	3.45 (3.12-3.80)	3.91 (3.52-4.30)	4.39 (3.93-4.82)	4.90 (4.37-5.39)	5.64 (4.98-6.20)	6.24 (5.47-6.88)
6-hr	2.13 (1.93-2.35)	2.53 (2.30-2.80)	3.10 (2.82-3.43)	3.58 (3.25-3.95)	4.26 (3.84-4.69)	4.83 (4.33-5.32)	5.43 (4.85-5.98)	6.09 (5.40-6.70)	7.01 (6.17-7.72)	7.78 (6.78-8.58)
12-hr	2.58 (2.36-2.85)	3.08 (2.82-3.40)	3.79 (3.46-4.18)	4.38 (3.99-4.82)	5.22 (4.73-5.74)	5.92 (5.34-6.51)	6.66 (5.97-7.32)	7.46 (6.65-8.20)	8.60 (7.59-9.45)	9.54 (8.36-10.5)
24-hr	3.14 (2.88-3.44)	3.76 (3.45-4.12)	4.64 (4.25-5.09)	5.37 (4.91-5.87)	6.39 (5.82-6.98)	7.23 (6.56-7.89)	8.12 (7.32-8.85)	9.06 (8.12-9.88)	10.4 (9.22-11.3)	11.5 (10.1-12.5)
2-day	3.75 (3.46-4.10)	4.49 (4.14-4.92)	5.53 (5.09-6.04)	6.36 (5.84-6.93)	7.52 (6.89-8.19)	8.46 (7.72-9.21)	9.45 (8.56-10.3)	10.5 (9.44-11.4)	11.9 (10.6-13.0)	13.1 (11.6-14.3)
3-day	4.01 (3.69-4.37)	4.80 (4.42-5.24)	5.89 (5.43-6.43)	6.77 (6.22-7.37)	7.98 (7.31-8.69)	8.96 (8.17-9.75)	9.97 (9.04-10.9)	11.0 (9.94-12.0)	12.5 (11.2-13.7)	13.6 (12.1-15.0)
4-day	4.26 (3.93-4.64)	5.11 (4.71-5.56)	6.26 (5.77-6.82)	7.18 (6.61-7.81)	8.45 (7.74-9.18)	9.46 (8.62-10.3)	10.5 (9.52-11.4)	11.6 (10.4-12.6)	13.0 (11.7-14.3)	14.2 (12.6-15.6)
7-day	5.06 (4.65-5.50)	6.06 (5.58-6.59)	7.43 (6.83-8.07)	8.52 (7.81-9.25)	10.0 (9.17-10.9)	11.3 (10.2-12.2)	12.6 (11.4-13.6)	13.9 (12.5-15.1)	15.7 (14.0-17.2)	17.2 (15.2-18.8)
10-day	5.77 (5.34-6.22)	6.90 (6.38-7.45)	8.39 (7.75-9.05)	9.55 (8.81-10.3)	11.1 (10.2-12.0)	12.4 (11.3-13.3)	13.6 (12.4-14.7)	14.9 (13.5-16.1)	16.6 (15.0-18.0)	18.0 (16.1-19.6)
20-day	7.88 (7.35-8.46)	9.36 (8.74-10.1)	11.1 (10.4-12.0)	12.5 (11.6-13.4)	14.2 (13.2-15.3)	15.5 (14.4-16.7)	16.8 (15.5-18.1)	18.1 (16.6-19.5)	19.7 (18.0-21.2)	20.9 (19.0-22.6)
30-day	9.63 (9.00-10.3)	11.4 (10.7-12.2)	13.4 (12.6-14.4)	15.0 (14.0-16.0)	16.9 (15.8-18.1)	18.4 (17.1-19.7)	19.9 (18.5-21.3)	21.4 (19.7-22.9)	23.2 (21.3-24.9)	24.6 (22.5-26.4)
45-day	12.1 (11.3-12.9)	14.3 (13.4-15.2)	16.7 (15.7-17.8)	18.5 (17.3-19.7)	20.8 (19.4-22.1)	22.4 (20.9-23.9)	24.1 (22.4-25.6)	25.6 (23.7-27.3)	27.6 (25.4-29.5)	29.0 (26.6-31.0)
60-day	14.5 (13.6-15.4)	17.1 (16.0-18.2)	19.8 (18.6-21.1)	21.8 (20.4-23.2)	24.3 (22.7-25.9)	26.1 (24.4-27.9)	27.8 (25.9-29.7)	29.5 (27.4-31.5)	31.4 (29.1-33.6)	32.9 (30.3-35.2)

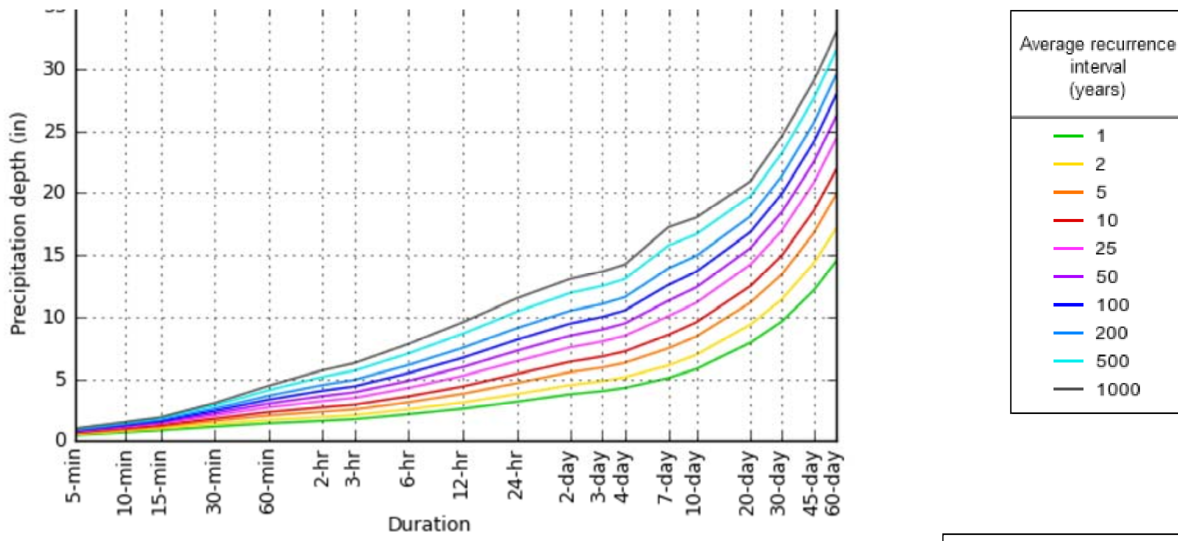
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

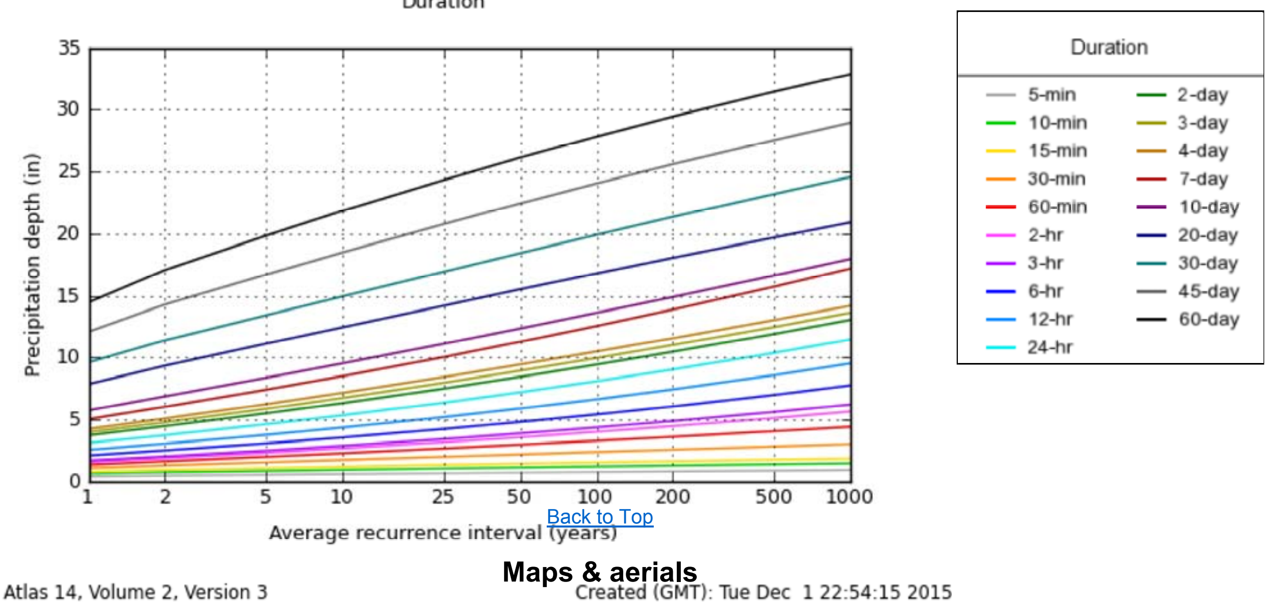
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical



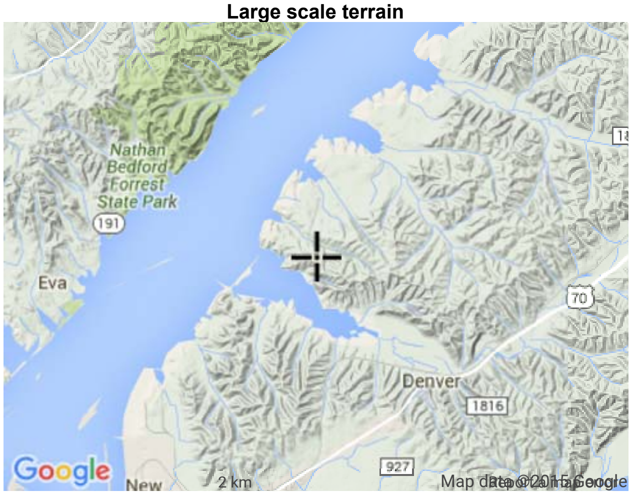


Back to Top

Maps & aerials

Small scale terrain

Large scale terrain



Large scale map

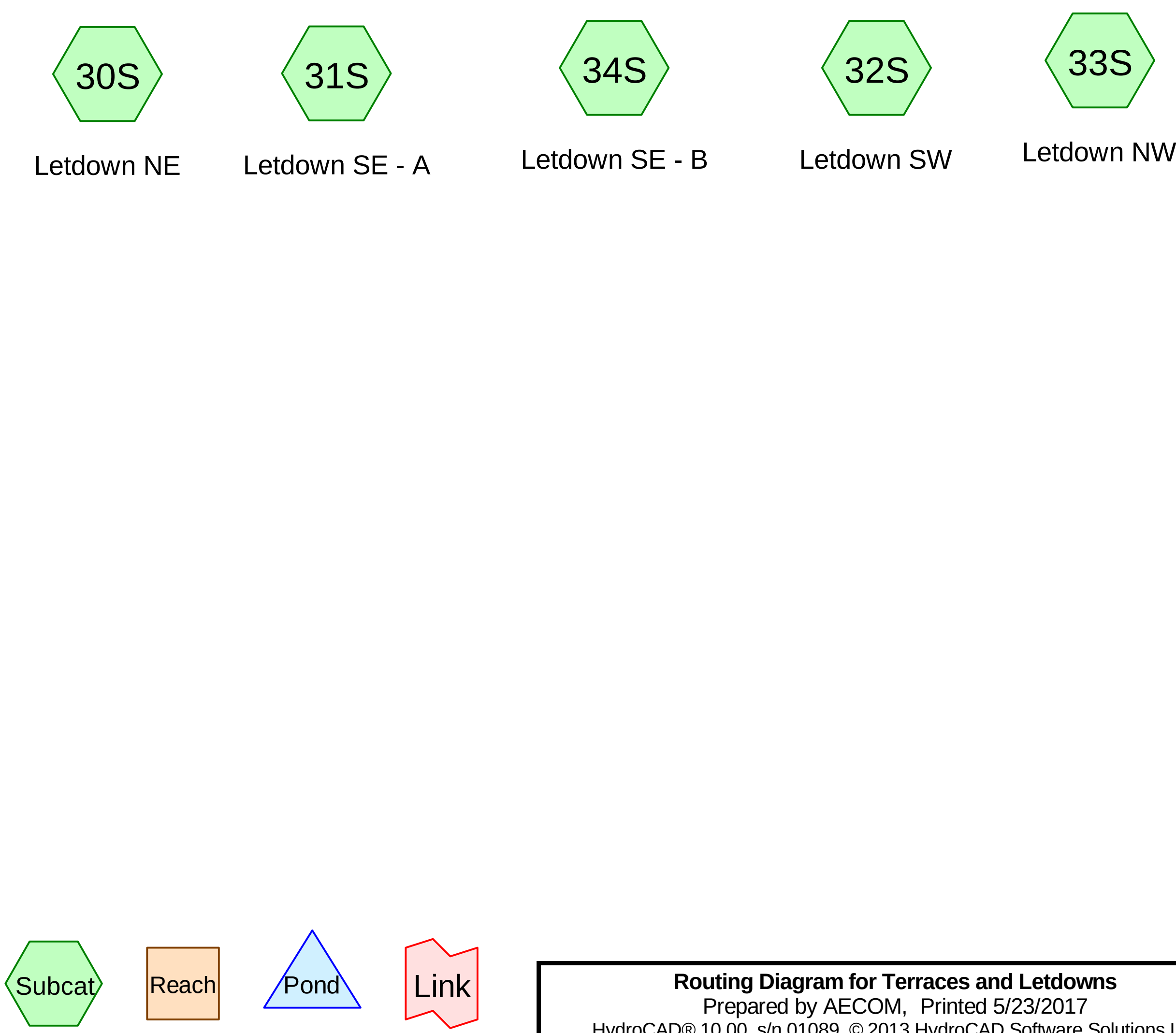
Large scale aerial

[Back to Top]

1.

ATTACHMENT B

Letdown Design



Routing Diagram for Terraces and Letdowns
Prepared by AECOM, Printed 5/23/2017

Summary for Subcatchment 30S: Letdown NE

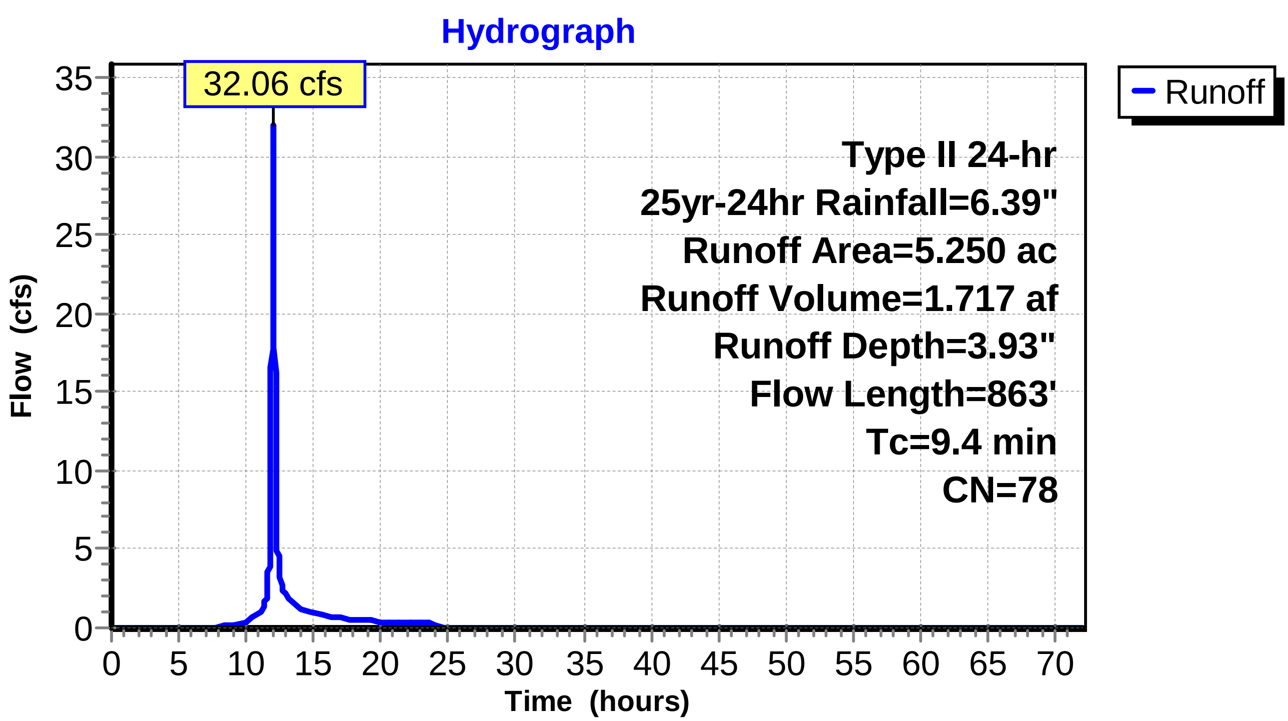
Runoff = 32.06 cfs @ 12.01 hrs, Volume= 1.717 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	5.250	78	
	5.250		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	72	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
3.2	285	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.3	452	0.1853	23.67	662.70	**Channel Flow, Letdown** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
9.4	863	Total			

Subcatchment 30S: Letdown NE



Hydrograph for Subcatchment 30S: Letdown NE

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.02
8.00	0.77	0.01	0.09
9.00	0.94	0.04	0.21
10.00	1.16	0.10	0.39
11.00	1.50	0.23	0.94
12.00	4.24	2.08	**31.93**
13.00	4.93	2.65	**1.96**
14.00	5.24	2.92	1.16
15.00	5.45	3.10	0.91
16.00	5.62	3.25	0.71
17.00	5.76	3.37	0.62
18.00	5.89	3.48	0.55
19.00	5.99	3.57	0.47
20.00	6.08	3.65	0.40
21.00	6.16	3.72	0.38
22.00	6.24	3.79	0.36
23.00	6.32	3.86	0.35
24.00	**6.39**	**3.93**	0.33
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 31S: Letdown SE - A

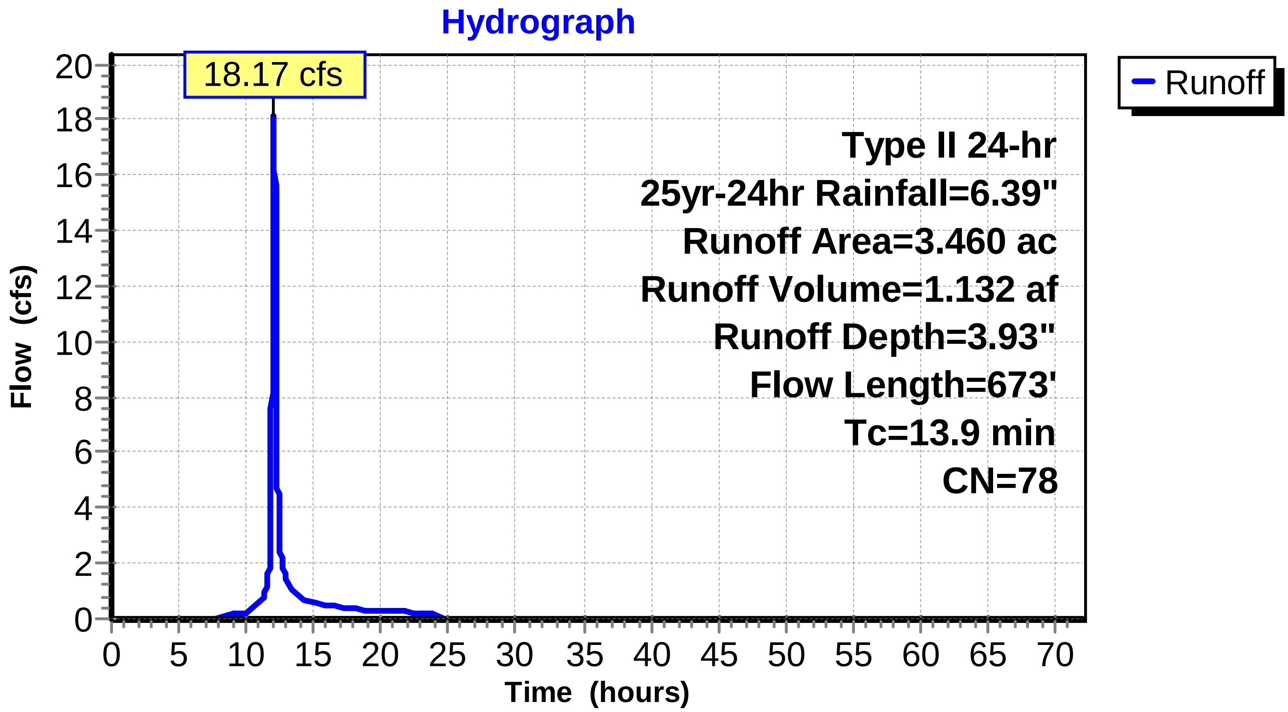
Runoff = 18.17 cfs @ 12.06 hrs, Volume= 1.132 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	3.460	78	
	3.460		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.4	90	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
4.3	390	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.1	93	0.0790	15.45	432.71	**Channel Flow, Letdown** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
13.9	673	Total			

Subcatchment 31S: Letdown SE - A



Hydrograph for Subcatchment 31S: Letdown SE - A

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.01
8.00	0.77	0.01	0.05
9.00	0.94	0.04	0.13
10.00	1.16	0.10	0.24
11.00	1.50	0.23	0.58
12.00	4.24	2.08	**15.97**
13.00	4.93	2.65	**1.36**
14.00	5.24	2.92	0.79
15.00	5.45	3.10	0.61
16.00	5.62	3.25	0.48
17.00	5.76	3.37	0.41
18.00	5.89	3.48	0.36
19.00	5.99	3.57	0.32
20.00	6.08	3.65	0.27
21.00	6.16	3.72	0.25
22.00	6.24	3.79	0.24
23.00	6.32	3.86	0.23
24.00	**6.39**	**3.93**	0.22
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 32S: Letdown SW

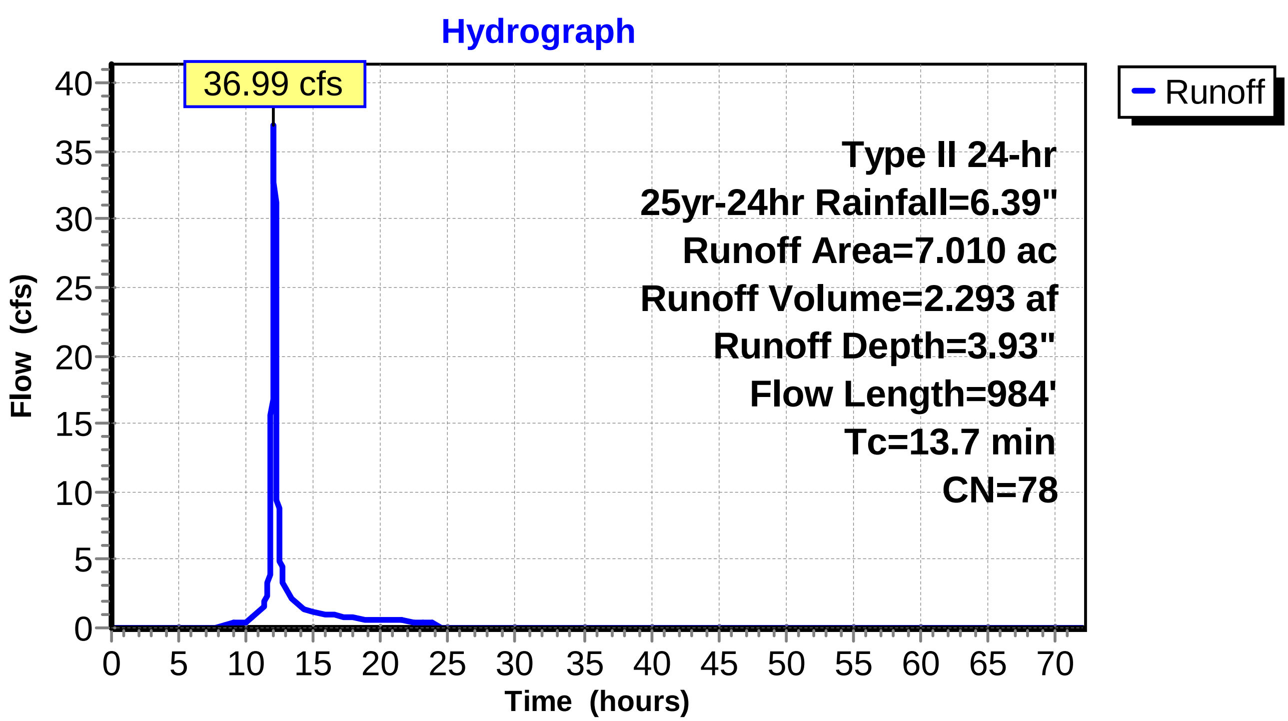
Runoff = 36.99 cfs @ 12.06 hrs, Volume= 2.293 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	7.010	78	
	7.010		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
9.1	100	0.0500	0.18		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.7	174	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
3.6	326	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.3	384	0.1866	23.75	665.02	**Channel Flow, Letdown** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
13.7	984	Total			

Subcatchment 32S: Letdown SW



Hydrograph for Subcatchment 32S: Letdown SW

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)	Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00	54.00	6.39	3.93	0.00
1.00	0.07	0.00	0.00	55.00	6.39	3.93	0.00
2.00	0.14	0.00	0.00	56.00	6.39	3.93	0.00
3.00	0.22	0.00	0.00	57.00	6.39	3.93	0.00
4.00	0.31	0.00	0.00	58.00	6.39	3.93	0.00
5.00	0.40	0.00	0.00	59.00	6.39	3.93	0.00
6.00	0.51	0.00	0.00	60.00	6.39	3.93	0.00
7.00	0.63	0.00	0.03	61.00	6.39	3.93	0.00
8.00	0.77	0.01	0.11	62.00	6.39	3.93	0.00
9.00	0.94	0.04	0.27	63.00	6.39	3.93	0.00
10.00	1.16	0.10	0.49	64.00	6.39	3.93	0.00
11.00	1.50	0.23	1.17	65.00	6.39	3.93	0.00
12.00	4.24	2.08	**32.80**	66.00	6.39	3.93	0.00
13.00	4.93	2.65	**2.75**	67.00	6.39	3.93	0.00
14.00	5.24	2.92	1.59	68.00	6.39	3.93	0.00
15.00	5.45	3.10	1.23	69.00	6.39	3.93	0.00
16.00	5.62	3.25	0.96	70.00	6.39	3.93	0.00
17.00	5.76	3.37	0.83	71.00	6.39	3.93	0.00
18.00	5.89	3.48	0.74	72.00	6.39	3.93	0.00
19.00	5.99	3.57	0.64				
20.00	6.08	3.65	0.54				
21.00	6.16	3.72	0.51				
22.00	6.24	3.79	0.49				
23.00	6.32	3.86	0.47				
24.00	**6.39**	**3.93**	0.45				
25.00	6.39	3.93	0.00				
26.00	6.39	3.93	0.00				
27.00	6.39	3.93	0.00				
28.00	6.39	3.93	0.00				
29.00	6.39	3.93	0.00				
30.00	6.39	3.93	0.00				
31.00	6.39	3.93	0.00				
32.00	6.39	3.93	0.00				
33.00	6.39	3.93	0.00				
34.00	6.39	3.93	0.00				
35.00	6.39	3.93	0.00				
36.00	6.39	3.93	0.00				
37.00	6.39	3.93	0.00				
38.00	6.39	3.93	0.00				
39.00	6.39	3.93	0.00				
40.00	6.39	3.93	0.00				
41.00	6.39	3.93	0.00				
42.00	6.39	3.93	0.00				
43.00	6.39	3.93	0.00				
44.00	6.39	3.93	0.00				
45.00	6.39	3.93	0.00				
46.00	6.39	3.93	0.00				
47.00	6.39	3.93	0.00				
48.00	6.39	3.93	0.00				
49.00	6.39	3.93	0.00				
50.00	6.39	3.93	0.00				
51.00	6.39	3.93	0.00				
52.00	6.39	3.93	0.00				
53.00	6.39	3.93	0.00				

Summary for Subcatchment 33S: Letdown NW

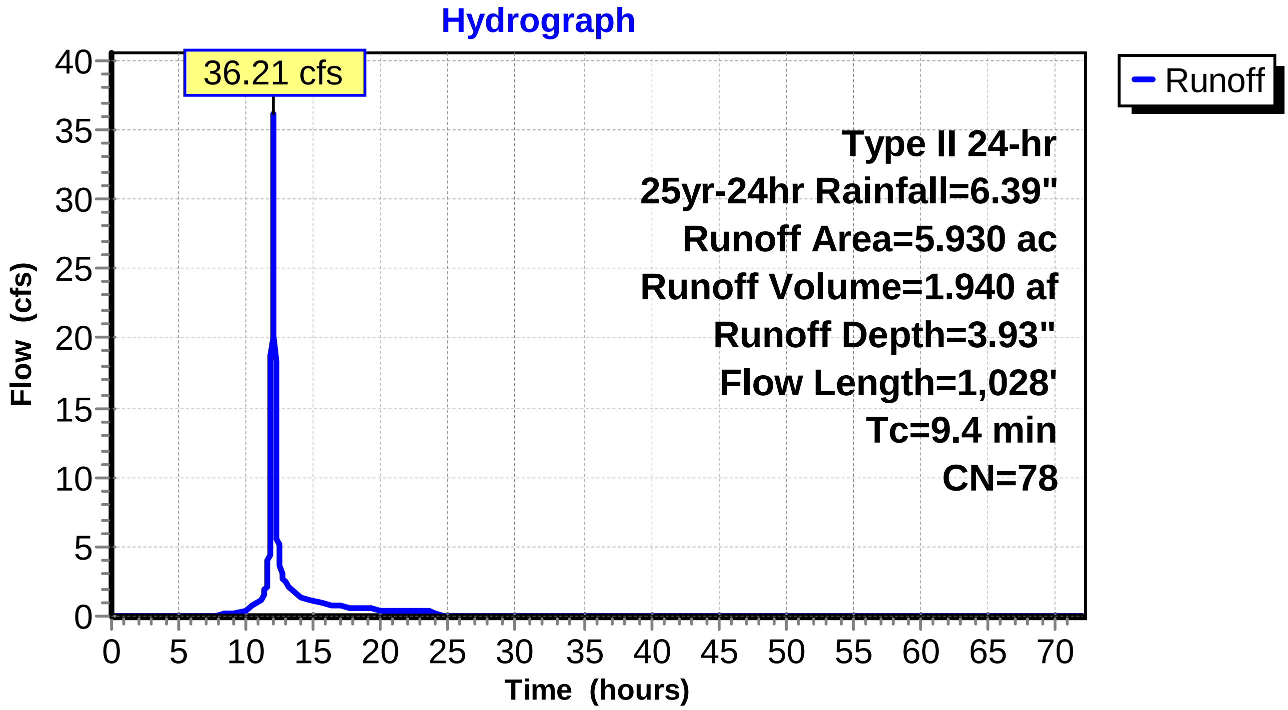
Runoff = 36.21 cfs @ 12.01 hrs, Volume= 1.940 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	5.930	78	
	5.930		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
3.4	76	0.3300	0.37		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
5.7	510	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
0.3	442	0.2090	25.14	703.80	**Channel Flow, Letdown** Area= 28.0 sf Perim= 19.0' r= 1.47' n= 0.035
9.4	1,028	Total			

Subcatchment 33S: Letdown NW



Hydrograph for Subcatchment 33S: Letdown NW

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.03
8.00	0.77	0.01	0.10
9.00	0.94	0.04	0.24
10.00	1.16	0.10	0.44
11.00	1.50	0.23	1.06
12.00	4.24	2.08	**36.07**
13.00	4.93	2.65	**2.21**
14.00	5.24	2.92	1.31
15.00	5.45	3.10	1.02
16.00	5.62	3.25	0.80
17.00	5.76	3.37	0.70
18.00	5.89	3.48	0.62
19.00	5.99	3.57	0.54
20.00	6.08	3.65	0.45
21.00	6.16	3.72	0.43
22.00	6.24	3.79	0.41
23.00	6.32	3.86	0.39
24.00	**6.39**	**3.93**	0.38
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Summary for Subcatchment 34S: Letdown SE - B

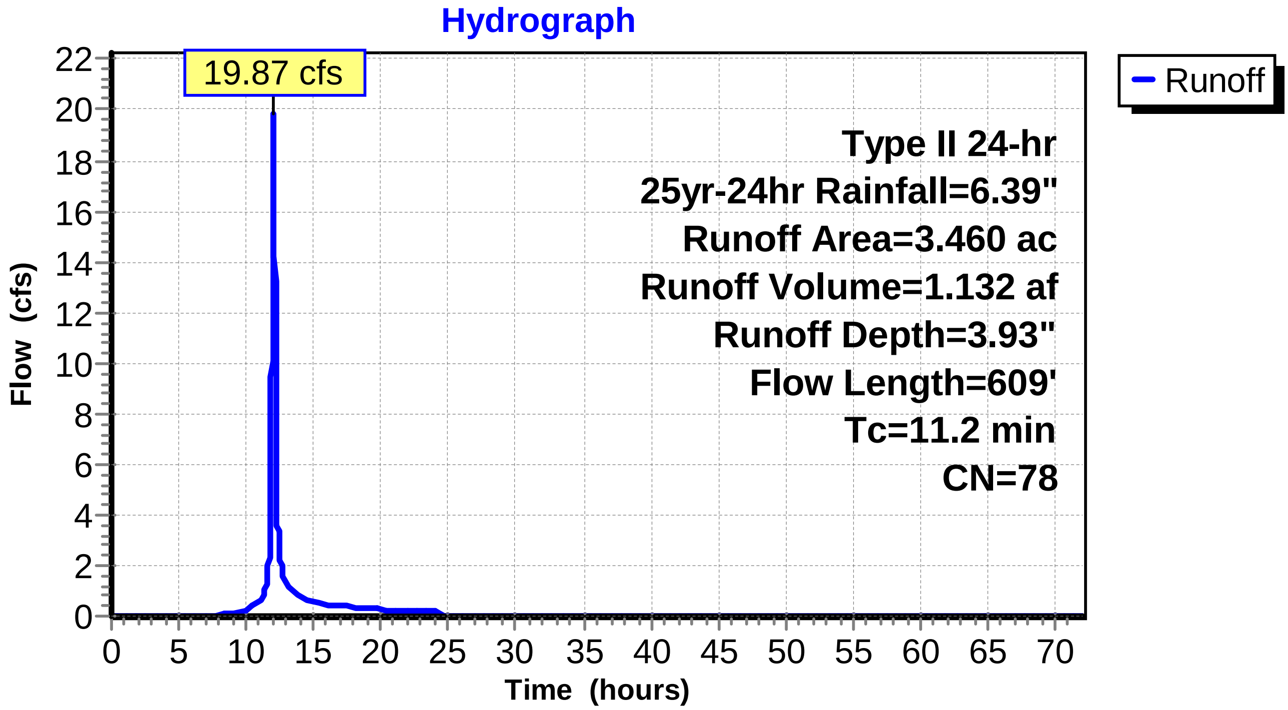
Runoff = 19.87 cfs @ 12.03 hrs, Volume= 1.132 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	3.460	78	
	3.460		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.6	54	0.0500	0.16		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.3	81	0.3300	4.02		**Shallow Concentrated Flow, Shallow Concentrated Flow** Short Grass Pasture Kv= 7.0 fps
5.3	474	0.0100	1.50		**Shallow Concentrated Flow, Terrace** Grassed Waterway Kv= 15.0 fps
11.2	609	Total			

Subcatchment 34S: Letdown SE - B



Hydrograph for Subcatchment 34S: Letdown SE - B

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.01
8.00	0.77	0.01	0.06
9.00	0.94	0.04	0.14
10.00	1.16	0.10	0.25
11.00	1.50	0.23	0.60
12.00	4.24	2.08	**19.07**
13.00	4.93	2.65	**1.32**
14.00	5.24	2.92	0.77
15.00	5.45	3.10	0.60
16.00	5.62	3.25	0.47
17.00	5.76	3.37	0.41
18.00	5.89	3.48	0.36
19.00	5.99	3.57	0.31
20.00	6.08	3.65	0.27
21.00	6.16	3.72	0.25
22.00	6.24	3.79	0.24
23.00	6.32	3.86	0.23
24.00	**6.39**	**3.93**	0.22
25.00	6.39	3.93	0.00
26.00	6.39	3.93	0.00
27.00	6.39	3.93	0.00
28.00	6.39	3.93	0.00
29.00	6.39	3.93	0.00
30.00	6.39	3.93	0.00
31.00	6.39	3.93	0.00
32.00	6.39	3.93	0.00
33.00	6.39	3.93	0.00
34.00	6.39	3.93	0.00
35.00	6.39	3.93	0.00
36.00	6.39	3.93	0.00
37.00	6.39	3.93	0.00
38.00	6.39	3.93	0.00
39.00	6.39	3.93	0.00
40.00	6.39	3.93	0.00
41.00	6.39	3.93	0.00
42.00	6.39	3.93	0.00
43.00	6.39	3.93	0.00
44.00	6.39	3.93	0.00
45.00	6.39	3.93	0.00
46.00	6.39	3.93	0.00
47.00	6.39	3.93	0.00
48.00	6.39	3.93	0.00
49.00	6.39	3.93	0.00
50.00	6.39	3.93	0.00
51.00	6.39	3.93	0.00
52.00	6.39	3.93	0.00
53.00	6.39	3.93	0.00
54.00	6.39	3.93	0.00
55.00	6.39	3.93	0.00
56.00	6.39	3.93	0.00
57.00	6.39	3.93	0.00
58.00	6.39	3.93	0.00
59.00	6.39	3.93	0.00
60.00	6.39	3.93	0.00
61.00	6.39	3.93	0.00
62.00	6.39	3.93	0.00
63.00	6.39	3.93	0.00
64.00	6.39	3.93	0.00
65.00	6.39	3.93	0.00
66.00	6.39	3.93	0.00
67.00	6.39	3.93	0.00
68.00	6.39	3.93	0.00
69.00	6.39	3.93	0.00
70.00	6.39	3.93	0.00
71.00	6.39	3.93	0.00
72.00	6.39	3.93	0.00

Hydraulic Analysis Report

Project Data

Project Title:

Designer:

Project Date: Thursday, June 16, 2016

Project Units: U.S. Customary Units

Notes:

Channel Analysis: Letdown - NW

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width: 10.0000 ft

Longitudinal Slope: 0.2090 ft/ft

Manning's n: 0.0350

Flow: 36.2100 cfs

Result Parameters

Depth: 0.3613 ft

Area of Flow: 3.8735 ft^2

Wetted Perimeter: 11.6156 ft

Hydraulic Radius: 0.3335 ft

Average Velocity: 9.3481 ft/s

Top Width: 11.4450 ft

Froude Number: 2.8317

Critical Depth: 0.7055 ft

Critical Velocity: 4.4982 ft/s

Critical Slope: 0.0216 ft/ft

Critical Top Width: 12.82 ft

Calculated Max Shear Stress: 4.7113 lb/ft^2

Calculated Avg Shear Stress: 4.3491 lb/ft^2

Channel Analysis: Letdown - NE

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width: 10.0000 ft

Longitudinal Slope: 0.1853 ft/ft

Manning's n: 0.0350

Flow: 32.0600 cfs

Result Parameters

Depth: 0.3485 ft

Area of Flow: 3.7283 ft^2

Wetted Perimeter: 11.5587 ft

Hydraulic Radius: 0.3226 ft

Average Velocity: 8.5991 ft/s

Top Width: 11.3941 ft

Froude Number: 2.6492

Critical Depth: 0.6529 ft

Critical Velocity: 4.3433 ft/s

Critical Slope: 0.0221 ft/ft

Critical Top Width: 12.61 ft

Calculated Max Shear Stress: 4.0300 lb/ft^2

Calculated Avg Shear Stress: 3.7296 lb/ft^2

Channel Analysis: Letdown - SE - A

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width: 10.0000 ft

Longitudinal Slope: 0.0791 ft/ft

Manning's n: 0.0350

Flow: 5.4100 cfs

Result Parameters

Depth: 0.1556 ft

Area of Flow: 1.6044 ft^2

Wetted Perimeter: 10.6959 ft

Hydraulic Radius: 0.1500 ft

Average Velocity: 3.3719 ft/s

Top Width: 10.6224 ft

Froude Number: 1.5290

Critical Depth: 0.2059 ft

Critical Velocity: 2.5239 ft/s

Critical Slope: 0.0310 ft/ft

Critical Top Width: 10.82 ft

Calculated Max Shear Stress: 0.7680 lb/ft^2

Calculated Avg Shear Stress: 0.7404 lb/ft^2

Channel Analysis: Letdown - SE - B

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width: 10.0000 ft

Longitudinal Slope: 0.1042 ft/ft

Manning's n: 0.0350

Flow: 5.4600 cfs

Result Parameters

Depth: 0.1443 ft

Area of Flow: 1.4847 ft^2

Wetted Perimeter: 10.6453 ft

Hydraulic Radius: 0.1395 ft

Average Velocity: 3.6775 ft/s

Top Width: 10.5772 ft

Froude Number: 1.7298

Critical Depth: 0.2071 ft

Critical Velocity: 2.5313 ft/s

Critical Slope: 0.0309 ft/ft

Critical Top Width: 10.83 ft

Calculated Max Shear Stress: 0.9383 lb/ft^2

Calculated Avg Shear Stress: 0.9068 lb/ft^2

Channel Analysis: Letdown - SW

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width: 10.0000 ft

Longitudinal Slope: 0.1866 ft/ft

Manning's n: 0.0350

Flow: 36.9900 cfs

Result Parameters

Depth: 0.3789 ft

Area of Flow: 4.0756 ft^2

Wetted Perimeter: 11.6943 ft

Hydraulic Radius: 0.3485 ft

Average Velocity: 9.0759 ft/s

Top Width: 11.5154 ft

Froude Number: 2.6884

Critical Depth: 0.7150 ft

Critical Velocity: 4.5259 ft/s

Critical Slope: 0.0215 ft/ft

Critical Top Width: 12.86 ft

Calculated Max Shear Stress: 4.4114 lb/ft^2

Calculated Avg Shear Stress: 4.0581 lb/ft^2

APPENDIX F.1E

Storm Water Terrace Design (Rev 3)

I. PURPOSE

The purpose of this analysis is to design the proposed landfill cap terrace channels for the Scepter Class II Disposal Facility in Waverly, TN consistent with state regulations.

II. SITE AND PROJECT DESCRIPTION

The terrace design was performed as part of the Part II Solid Waste Permit Application. The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management. Refer to Exhibit 1 for a depiction of the terrace locations in relation to the landfill.

The following sections summarize the design criteria, procedure, assumptions, and results of the terrace design.

III. REGULATORY REQUIREMENTS / DESIGN CRITERIA

The below parts of the TDEC regulations specify requirements for the design of the terraces:

Rule 0400-11-01-.04(2)(i)2.

The operator must design, construct, operate, and maintain a run-off management system to collect and control at least the peak flow resulting from a 24-hour, 25 year storm.

The results of the analysis presented herein show that the terraces are designed to collect and control at least the peak flow resulting from a 25-year/24-hour storm. The terrace channels will convey landfill run-off flow towards the letdown channels and perimeter ditch.

Rule 0400-11-01-.04(2)(i)6.

The operator must take other erosion control measure (e.g., temporary mulching or seeding, silt barriers) as necessary to control erosion of the site.

Proper channel lining will be selected based on anticipated velocities such that erosion within the terrace channels will be significantly reduced.

IV. PROCEDURE

Design of the landfill site stormwater features was an iterative process beginning with basic assumptions and a proposed grading plan for the site. The hydraulic features of the terraces were initially assumed and then confirmed through analysis.

The AutoCAD Civil 3D software package was used to generate the proposed site grading plan and subsequently to determine drainage areas, volumes, and other site geometry. HydroCAD (version 10.00) modeling software was used to conduct the hydrologic calculations. The Federal Highway Administration's Hydraulic Toolbox was used to conduct hydraulic calculations for this analysis with inputs based on the site geometry, rainfall data, and other design assumptions.

The presumed worst case terraces was modeled based on drainage area/length and channel slope.

The HydroCAD model was used to generate peak flow rates for the upstream watershed features. Hydraulic Toolbox was used to generate channel velocities and water surface elevations at the most critical points along the letdown alignment. The proposed channels will pass flows equal to and lesser than those generated by the 24-hour, 25-year storm.

V. NOTES/ASSUMPTIONS

The following is a list of key notes and assumptions made in completing this analysis.

- The terraces were designed based on the worst case velocity and depth among the selected candidates in order to be conservative.
- A design parameter minimum six (6) inches of freeboard was used.
- Within the HydroCAD program, the runoff was calculated using the SCS TR-20 method.
- Runoff curve numbers (CN) used in the analysis were as follows:
    - 78 for landfill vegetated final cap cover and offsite areas,
    - 98 for sediment basin water surface.
    - 77 for exposed and disturbed soil (daily cover); however exposed and disturbed soil was ignored to be conservative and 78 was used within landfill construction footprint.
- The time of concentration was calculated using the Curve Number Method in HydroCAD which takes inputs for each drainage area of the longest hydraulic flow path and average land slope.

VI. SUMMARY OF RESULTS

The terraces properly control and convey the water volume generated by the 24-hour, 25-year storm such that the average flow depth stays safely below the impounding sideslopes with much greater than six (6) inches of freeboard. A grass vegetated channel was chosen for all terraces as the peak velocity does not exceed 2.5 fps.

VII. CONCLUSIONS

The proposed grading of the landfill site in combination with the design of the terraces as presented above is sufficient to safely control and convey the 24-hour, 25-year storm as stipulated by TDEC regulations. Calculations for proper conveyance of upstream and downstream flows were not presented in this analysis and are performed separately. Refer to accompanying calculations for upstream and downstream conveyance design features (storm water letdowns, ditches, culverts and sediment basins).

VIII. ATTACHMENTS

Figures:
Exhibit 1: Drainage Map

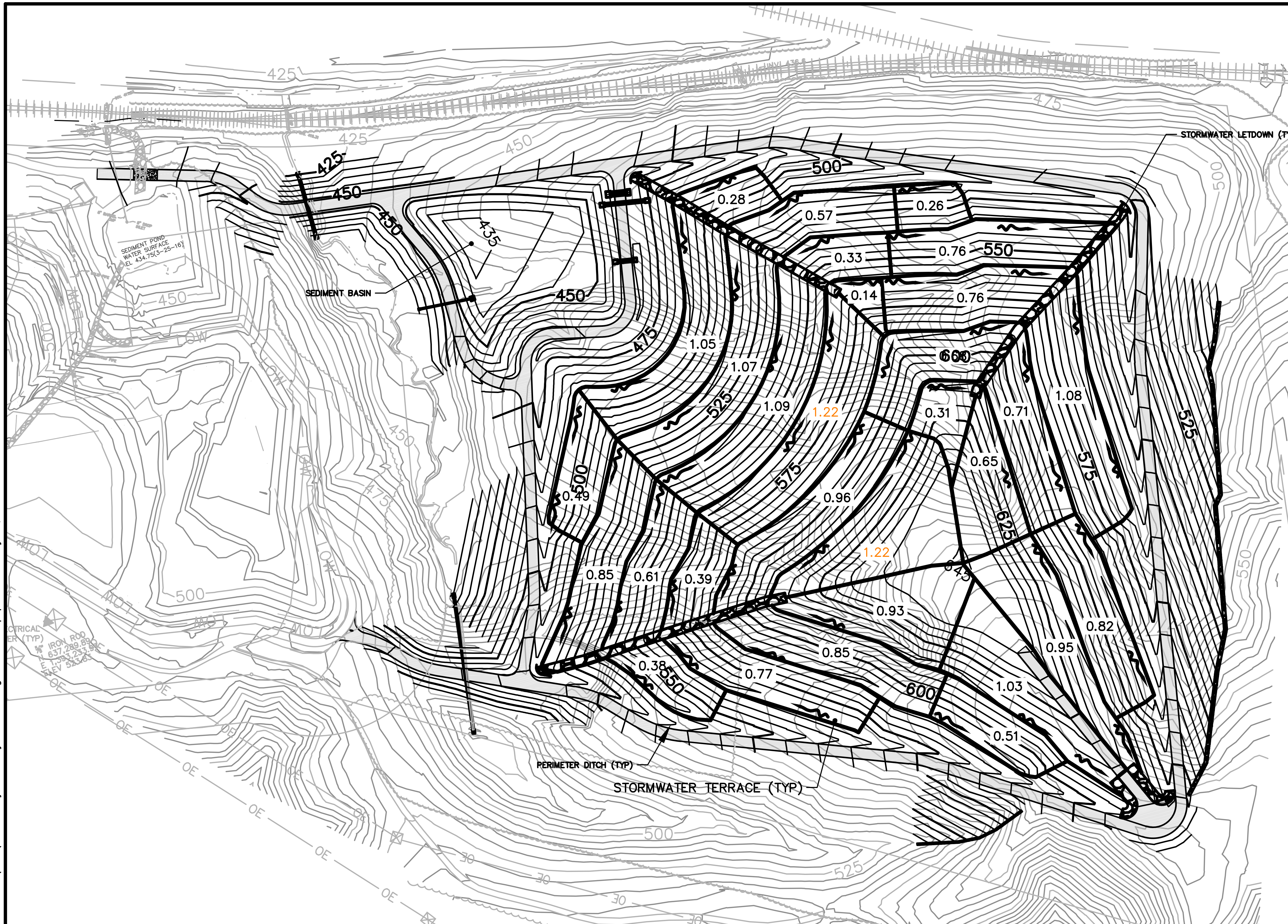
Attachments:
Attachment A: NOAA Precipitation Frequency Data
Attachment B: Terrace: Drainage Map, HydroCAD report, and Hydraulic Toolbox Report

IX. REFERENCES

1. TDEC, *"Rules of Tennessee Department of Environment and Conservation, Chapter 0400-11-01 – Solid Waste Processing and Disposal"*, Solid Waste Management, September 2012.

EXHIBIT 1

Drainage Map



DRAINAGE AREAS

MAX. AREA = 1.22 ACRES
CN = 78
TC = 5.0 MINS

LEGEND

- ▲ BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (25FT)
- EXISTING GRADE MINOR CONTOUR (5FT)
- FINAL GRADE MAJOR CONTOUR (25FT)
- FINAL GRADE MINOR CONTOUR (5FT)
- STORMWATER FLOW DIRECTION
- 1.22 TERRACE DRAINAGE AREA (ACRES)

AECOM

SCEPTER, INC.

WAVERLY, TENNESSEE

TERRACE DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/9/17	1

S:\2016\Scepter\028 Waverly\02 Landfill\CAD\x-tributary boundary-letdown.dwg User:nick.popkowski May 23, 2017 - 8:22am

ATTACHMENT A

NOAA Precipitation Frequency Data

NOAA Atlas 14, Volume 2, Version 3
Location name: Waverly, Tennessee, US*
Latitude: 36.0689°, Longitude: -87.9462°
Elevation: 523 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.405 (0.368–0.450)	0.476 (0.434–0.529)	0.547 (0.497–0.607)	0.604 (0.548–0.668)	0.675 (0.610–0.746)	0.728 (0.655–0.803)	0.781 (0.699–0.862)	0.832 (0.742–0.917)	0.898 (0.794–0.990)	0.948 (0.833–1.05)
10-min	0.647 (0.588–0.718)	0.761 (0.694–0.845)	0.876 (0.796–0.972)	0.965 (0.877–1.07)	1.08 (0.972–1.19)	1.16 (1.04–1.28)	1.24 (1.11–1.37)	1.32 (1.18–1.45)	1.42 (1.26–1.57)	1.49 (1.31–1.65)
15-min	0.809 (0.735–0.898)	0.957 (0.872–1.06)	1.11 (1.01–1.23)	1.22 (1.11–1.35)	1.36 (1.23–1.51)	1.47 (1.32–1.62)	1.57 (1.40–1.73)	1.66 (1.48–1.83)	1.79 (1.58–1.97)	1.88 (1.65–2.07)
30-min	1.11 (1.01–1.23)	1.32 (1.20–1.47)	1.58 (1.43–1.75)	1.77 (1.61–1.96)	2.02 (1.83–2.23)	2.21 (1.99–2.44)	2.40 (2.15–2.65)	2.59 (2.31–2.86)	2.84 (2.51–3.14)	3.04 (2.67–3.35)
60-min	1.38 (1.26–1.54)	1.66 (1.51–1.84)	2.02 (1.83–2.24)	2.30 (2.09–2.55)	2.69 (2.43–2.97)	2.99 (2.70–3.31)	3.31 (2.96–3.65)	3.63 (3.24–4.01)	4.08 (3.61–4.50)	4.43 (3.89–4.89)
2-hr	1.59 (1.44–1.76)	1.91 (1.73–2.11)	2.33 (2.11–2.59)	2.69 (2.43–2.98)	3.19 (2.87–3.52)	3.60 (3.22–3.98)	4.03 (3.59–4.46)	4.49 (3.98–4.97)	5.14 (4.52–5.70)	5.67 (4.95–6.29)
3-hr	1.72 (1.57–1.89)	2.05 (1.87–2.27)	2.52 (2.29–2.78)	2.91 (2.64–3.20)	3.45 (3.12–3.80)	3.91 (3.52–4.30)	4.39 (3.93–4.82)	4.90 (4.37–5.39)	5.64 (4.98–6.20)	6.24 (5.47–6.88)
6-hr	2.13 (1.93–2.35)	2.53 (2.30–2.80)	3.10 (2.82–3.43)	3.58 (3.25–3.95)	4.26 (3.84–4.69)	4.83 (4.33–5.32)	5.43 (4.85–5.98)	6.09 (5.40–6.70)	7.01 (6.17–7.72)	7.78 (6.78–8.58)
12-hr	2.58 (2.36–2.85)	3.08 (2.82–3.40)	3.79 (3.46–4.18)	4.38 (3.99–4.82)	5.22 (4.73–5.74)	5.92 (5.34–6.51)	6.66 (5.97–7.32)	7.46 (6.65–8.20)	8.60 (7.59–9.45)	9.54 (8.36–10.5)
24-hr	3.14 (2.88–3.44)	3.76 (3.45–4.12)	4.64 (4.25–5.09)	5.37 (4.91–5.87)	6.39 (5.82–6.98)	7.23 (6.56–7.89)	8.12 (7.32–8.85)	9.06 (8.12–9.88)	10.4 (9.22–11.3)	11.5 (10.1–12.5)
2-day	3.75 (3.46–4.10)	4.49 (4.14–4.92)	5.53 (5.09–6.04)	6.36 (5.84–6.93)	7.52 (6.89–8.19)	8.46 (7.72–9.21)	9.45 (8.56–10.3)	10.5 (9.44–11.4)	11.9 (10.6–13.0)	13.1 (11.6–14.3)
3-day	4.01 (3.69–4.37)	4.80 (4.42–5.24)	5.89 (5.43–6.43)	6.77 (6.22–7.37)	7.98 (7.31–8.69)	8.96 (8.17–9.75)	9.97 (9.04–10.9)	11.0 (9.94–12.0)	12.5 (11.2–13.7)	13.6 (12.1–15.0)
4-day	4.26 (3.93–4.64)	5.11 (4.71–5.56)	6.26 (5.77–6.82)	7.18 (6.61–7.81)	8.45 (7.74–9.18)	9.46 (8.62–10.3)	10.5 (9.52–11.4)	11.6 (10.4–12.6)	13.0 (11.7–14.3)	14.2 (12.6–15.6)
7-day	5.06 (4.65–5.50)	6.06 (5.58–6.59)	7.43 (6.83–8.07)	8.52 (7.81–9.25)	10.0 (9.17–10.9)	11.3 (10.2–12.2)	12.6 (11.4–13.6)	13.9 (12.5–15.1)	15.7 (14.0–17.2)	17.2 (15.2–18.8)
10-day	5.77 (5.34–6.22)	6.90 (6.38–7.45)	8.39 (7.75–9.05)	9.55 (8.81–10.3)	11.1 (10.2–12.0)	12.4 (11.3–13.3)	13.6 (12.4–14.7)	14.9 (13.5–16.1)	16.6 (15.0–18.0)	18.0 (16.1–19.6)
20-day	7.88 (7.35–8.46)	9.36 (8.74–10.1)	11.1 (10.4–12.0)	12.5 (11.6–13.4)	14.2 (13.2–15.3)	15.5 (14.4–16.7)	16.8 (15.5–18.1)	18.1 (16.6–19.5)	19.7 (18.0–21.2)	20.9 (19.0–22.6)
30-day	9.63 (9.00–10.3)	11.4 (10.7–12.2)	13.4 (12.6–14.4)	15.0 (14.0–16.0)	16.9 (15.8–18.1)	18.4 (17.1–19.7)	19.9 (18.5–21.3)	21.4 (19.7–22.9)	23.2 (21.3–24.9)	24.6 (22.5–26.4)
45-day	12.1 (11.3–12.9)	14.3 (13.4–15.2)	16.7 (15.7–17.8)	18.5 (17.3–19.7)	20.8 (19.4–22.1)	22.4 (20.9–23.9)	24.1 (22.4–25.6)	25.6 (23.7–27.3)	27.6 (25.4–29.5)	29.0 (26.6–31.0)
60-day	14.5 (13.6–15.4)	17.1 (16.0–18.2)	19.8 (18.6–21.1)	21.8 (20.4–23.2)	24.3 (22.7–25.9)	26.1 (24.4–27.9)	27.8 (25.9–29.7)	29.5 (27.4–31.5)	31.4 (29.1–33.6)	32.9 (30.3–35.2)

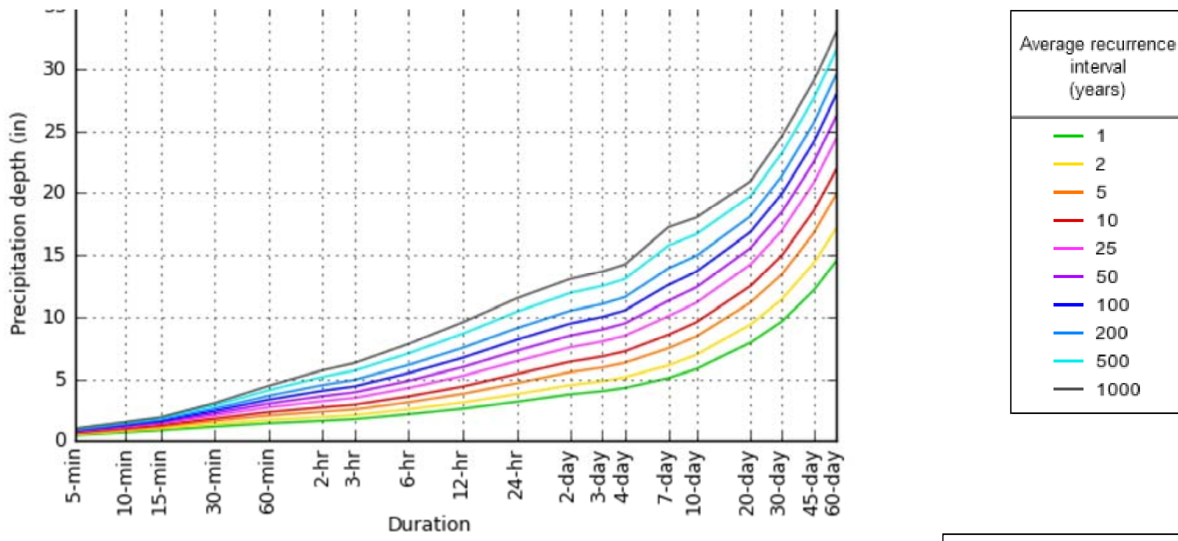
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

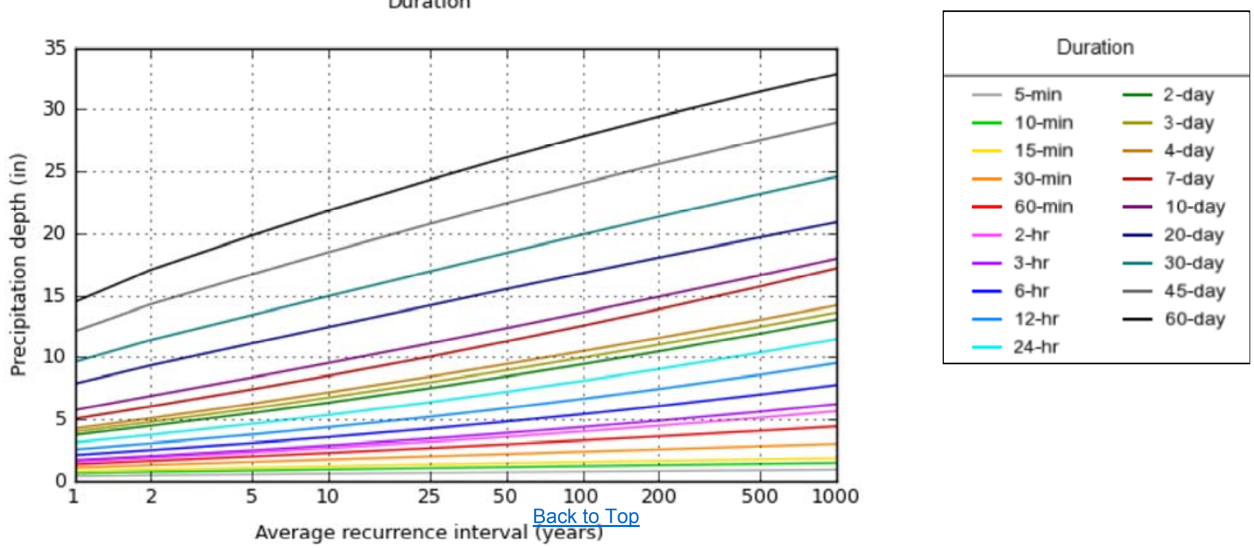
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical





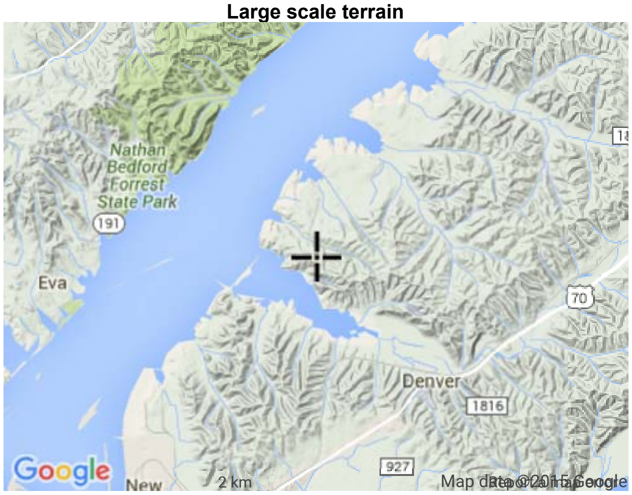
Back to Top

NOAA Atlas 14, Volume 2, Version 3 Created (GMT): Tue Dec 1 22:54:15 2015

Maps & aerials

Small scale terrain

Large scale terrain



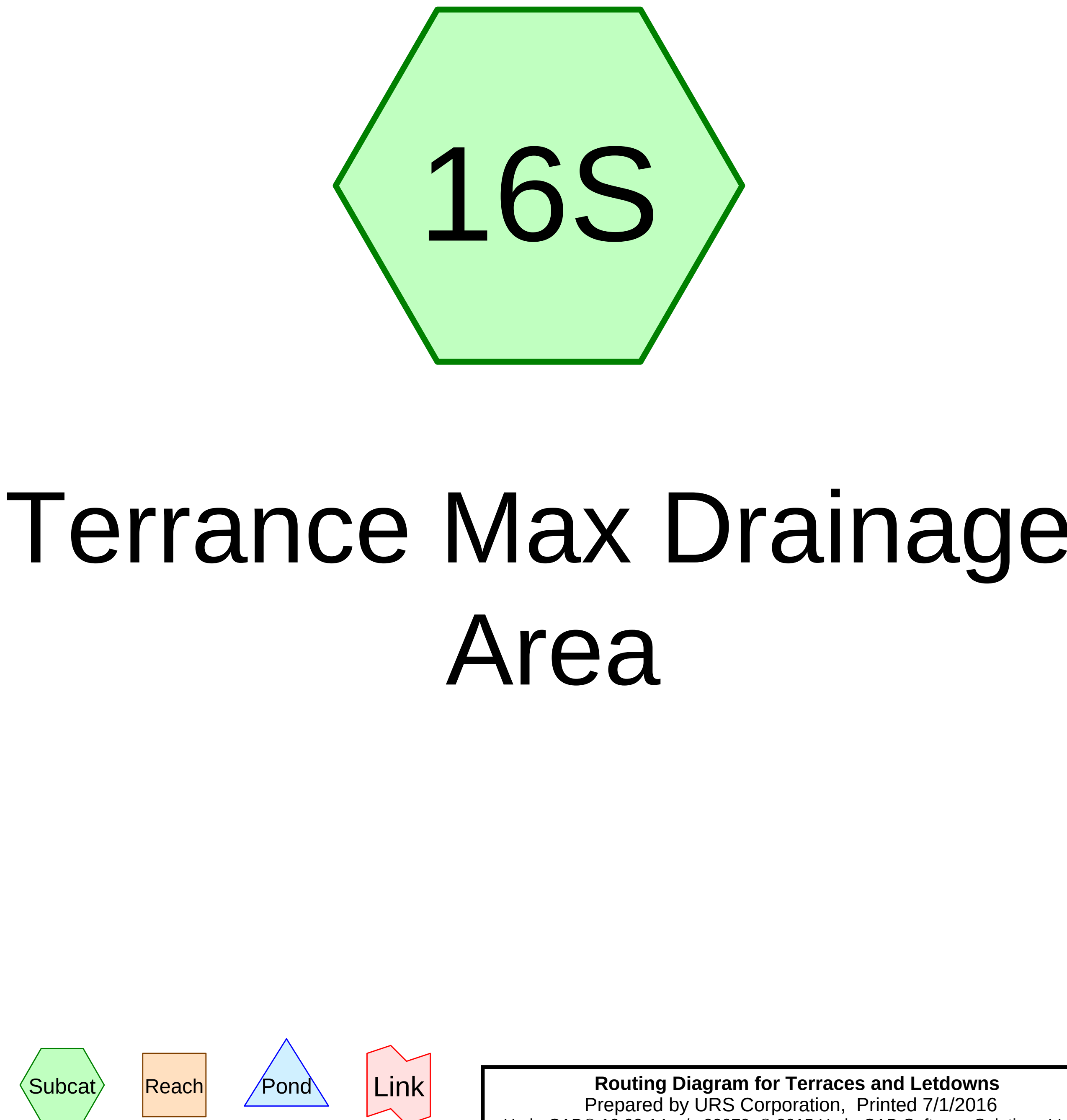
Large scale map

Large scale aerial

[Back to Top]

ATTACHMENT B

Terrace Design



Routing Diagram for Terraces and Letdowns
Prepared by URS Corporation, Printed 7/1/2016

Summary for Subcatchment 16S: Terrance Max Drainage Area

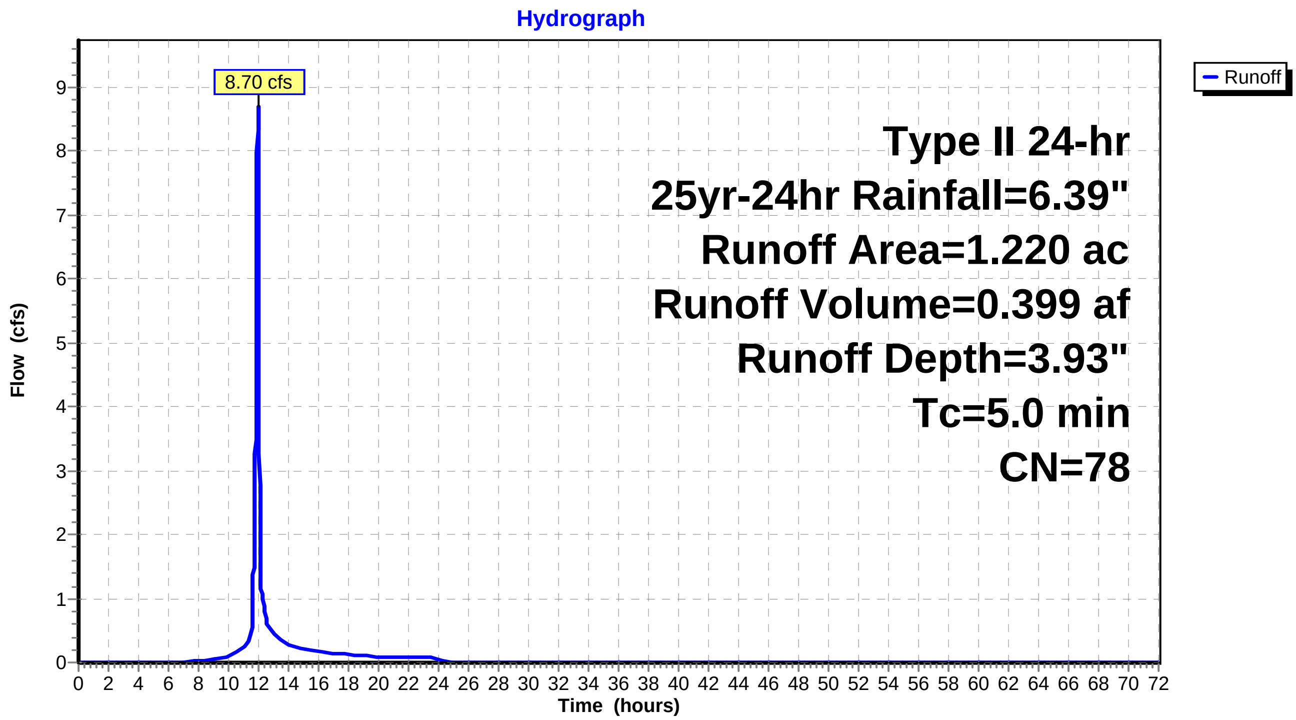
Runoff = 8.70 cfs @ 11.96 hrs, Volume= 0.399 af, Depth= 3.93"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	1.220	78	>75% Grass cover, Good, HSG D
	1.220		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry, Min. Time of Concentration**

Subcatchment 16S: Terrance Max Drainage Area



Hydrograph for Subcatchment 16S: Terrance Max Drainage Area

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)	Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00	53.00	6.39	3.93	0.00
1.00	0.07	0.00	0.00	54.00	6.39	3.93	0.00
2.00	0.14	0.00	0.00	55.00	6.39	3.93	0.00
3.00	0.22	0.00	0.00	56.00	6.39	3.93	0.00
4.00	0.31	0.00	0.00	57.00	6.39	3.93	0.00
5.00	0.40	0.00	0.00	58.00	6.39	3.93	0.00
6.00	0.51	0.00	0.00	59.00	6.39	3.93	0.00
7.00	0.63	0.00	0.01	60.00	6.39	3.93	0.00
8.00	0.77	0.01	0.02	61.00	6.39	3.93	0.00
9.00	0.94	0.04	0.05	62.00	6.39	3.93	0.00
10.00	1.16	0.10	0.09	63.00	6.39	3.93	0.00
11.00	1.50	0.23	**0.23**	64.00	6.39	3.93	0.00
12.00	4.24	2.08	**7.39**	65.00	6.39	3.93	0.00
13.00	4.93	2.65	0.43	66.00	6.39	3.93	0.00
14.00	5.24	2.92	0.26	67.00	6.39	3.93	0.00
15.00	5.45	3.10	0.21	68.00	6.39	3.93	0.00
16.00	5.62	3.25	0.16	69.00	6.39	3.93	0.00
17.00	5.76	3.37	0.14	70.00	6.39	3.93	0.00
18.00	5.89	3.48	0.13	71.00	6.39	3.93	0.00
19.00	5.99	3.57	0.11	72.00	6.39	3.93	0.00
20.00	6.08	3.65	0.09				
21.00	6.16	3.72	0.09				
22.00	6.24	3.79	0.08				
23.00	6.32	3.86	0.08				
24.00	**6.39**	**3.93**	0.08				
25.00	6.39	3.93	0.00				
26.00	6.39	3.93	0.00				
27.00	6.39	3.93	0.00				
28.00	6.39	3.93	0.00				
29.00	6.39	3.93	0.00				
30.00	6.39	3.93	0.00				
31.00	6.39	3.93	0.00				
32.00	6.39	3.93	0.00				
33.00	6.39	3.93	0.00				
34.00	6.39	3.93	0.00				
35.00	6.39	3.93	0.00				
36.00	6.39	3.93	0.00				
37.00	6.39	3.93	0.00				
38.00	6.39	3.93	0.00				
39.00	6.39	3.93	0.00				
40.00	6.39	3.93	0.00				
41.00	6.39	3.93	0.00				
42.00	6.39	3.93	0.00				
43.00	6.39	3.93	0.00				
44.00	6.39	3.93	0.00				
45.00	6.39	3.93	0.00				
46.00	6.39	3.93	0.00				
47.00	6.39	3.93	0.00				
48.00	6.39	3.93	0.00				
49.00	6.39	3.93	0.00				
50.00	6.39	3.93	0.00				
51.00	6.39	3.93	0.00				
52.00	6.39	3.93	0.00				

Hydraulic Analysis Report

Project Data

Project Title:

Designer:

Project Date: Thursday, June 16, 2016

Project Units: U.S. Customary Units

Notes:

Channel Analysis: Terrace (Max Water Depth)

Notes:

Input Parameters

Channel Type: Custom Cross Section

Cross Section Data

Elevation (ft)	Elevation (ft)	Manning's n
0.00	0.00	0.0350
15.00	-1.50	0.0350
19.50	0.00	-----

Longitudinal Slope: 0.0100 ft/ft

Flow: 8.7000 cfs

Result Parameters

Depth: 0.7745 ft

Area of Flow: 3.8988 ft^2

Wetted Perimeter: 10.2326 ft

Hydraulic Radius: 0.3810 ft

Average Velocity: 2.2314 ft/s

Top Width: 10.0683 ft

Froude Number: 0.6319

Critical Depth: 0.6446 ft

Critical Velocity: 3.2215 ft/s

Critical Slope: 0.0266 ft/ft

Critical Top Width: 8.38 ft

Calculated Max Shear Stress: 0.4833 lb/ft^2

Calculated Avg Shear Stress: 0.2378 lb/ft^2

Composite Manning's n Equation: Lotter method

Manning's n: 0.0350

APPENDIX F.1F

Storm Water Terrace Spacing (Rev 3)

I. <u>PURPOSE</u>

To verify that proposed storm water terraces are spaced such that the predicted soil erosion from the final cover of the Scepter East Phase Facility is less than 5 tons per acre per year. The following sections summarize the methodology, assumptions, and results of the soil erosion analysis. For further detail on the specific calculations performed, refer to the corresponding input/output data provided in the attachments.

II. <u>METHOD</u>

The intent of this calculation is to analyze and verify that predicted soil erosion from the final cover system of the Scepter East Phase Facility will be less than 5 tons per acre per year. The Revised Universal Soil Loss Equation (RUSLE) will be used to estimate the soil loss from interrill (sheet) and rill erosion caused by rainfall and associated overland flow. RUSLE is an empirical equation which includes several factors based on long-term averages. The equation is expressed as follows:

$$E = R * K * LS * C * P$$

Where:

E	=	Computed Soil Loss in tons/acre/year
R	=	Rainfall Energy Factor (Erosivity Index)
K	=	Soil Erodibility Factor
LS	=	Topographic Factor
C	=	Crop Management Factor
P	=	Support Practice Factor

It should be noted that the soil loss calculated by RUSLE is the rate of soil from the landscape profile analyzed (the slope length), not the amount of sediment leaving a field or watershed. RUSLE cannot be used to estimate or predict soil loss from individual storms or from a particular year of weather condition.

The following provides a description of each factor utilized in the analysis:

<u>Rainfall Energy Factor (R)</u>: The “R” factor represents the erosivity of the climate at a particular location. An average annual value of “R” is determined from historical weather records and is the average annual sum of the erosivity of individual storm. “R” factors may be chosen using the following figure (Reference 1):

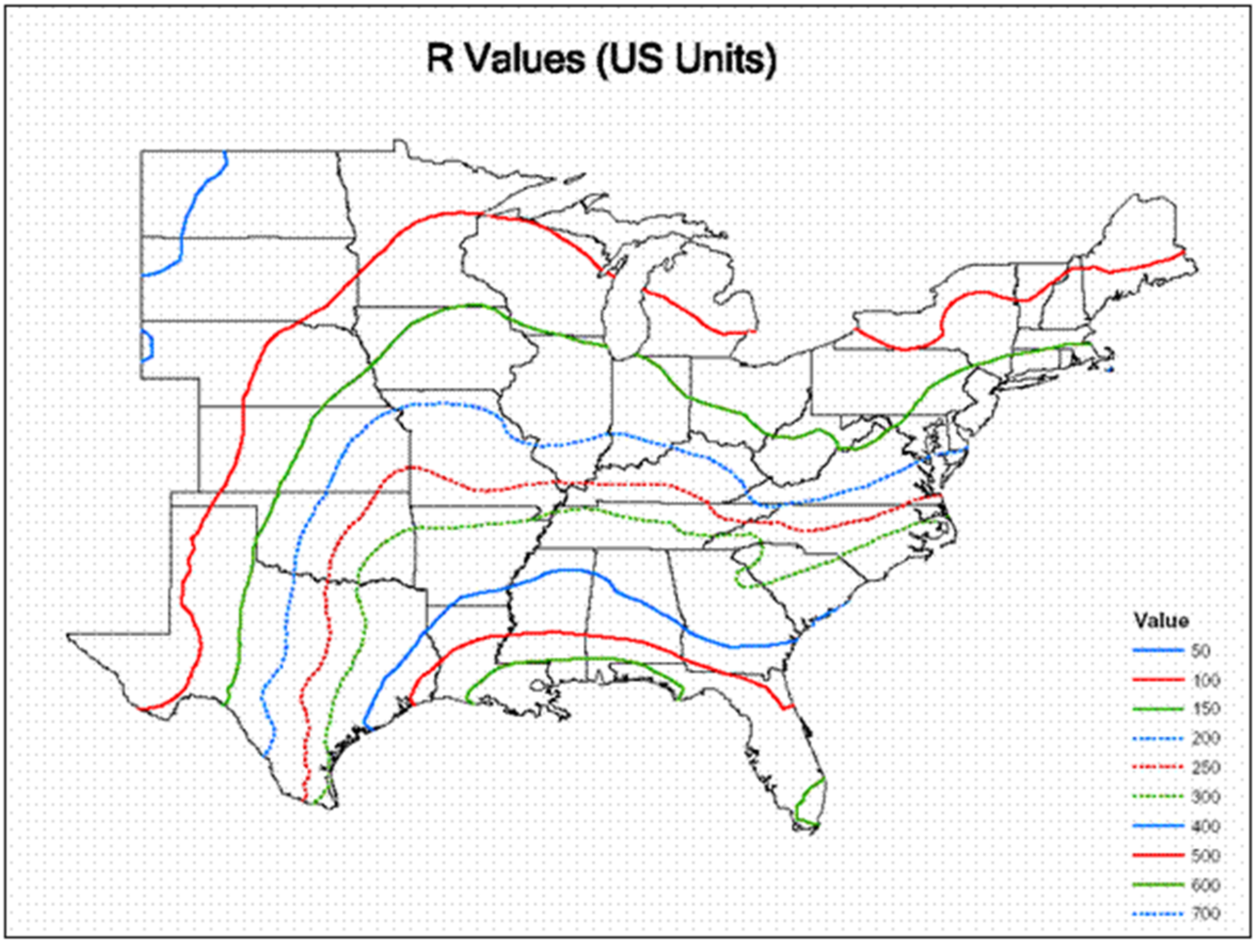


Figure 1. Average Annual "R" Factor

An "R" factor of 300 was chosen based on the location of the Scepter Inc. Disposal Facility in Waverly, Tennessee.

Soil Erodibility Factor (K): The "K" factor represents the measurement of soil erodibility as affected by intrinsic soil properties. Typical "K" factors may be chosen from the following table (Reference 2).

Table 1. Soil Erodibility Factor "K"[(a)]

	P_{OM}(%)		
Textural Class	<0.5	2	4
Sand	0.05	0.03	0.02
Fine sand	0.16	0.14	0.10
Very fine sand	0.42	0.36	0.28
Loamy sand	0.12	0.10	0.08
Loamy fine sand	0.24	0.20	0.16
Loamy very fine sand	0.44	0.38	0.30
Sandy loam	0.27	0.24	0.19
Fine sandy loam	0.35	0.30	0.24
Very fine sandy loam	0.47	0.41	0.33
Loam	0.38	0.34	0.29
Silt loam	0.48	0.42	0.33
Silt	0.60	0.52	0.42
Sandy clay loam	0.27	0.25	0.21
Clay loam	0.28	0.25	0.21
Silty clay loam	0.37	0.32	0.26
Sandy clay	0.14	0.13	0.12
Silty clay	0.25	**0.23**	0.19
Clay		0.13-0.2	

(a) The values shown are estimated averages of broad ranges of specific soil values. When a texture is near the border line of two texture classes, use the average of the two K_{fact} values. In addition, the values shown are commensurate with the English units used in the cited reference (and as used in the source-term module input files).

A "K" factor of 0.23 was used, which represents average Silty clay, and should conservatively model typical soil used at this site.

Topographic Factor (LS): The Topographic Factor "LS" is a measure of sediment production, measured as a function of the Slope Length Factor "L" and the Slope Steepness Factor "S". The topographic factor is calculated as follows (Reference 3, Equations 4-1, 4-2, 4-3, and 4-5):

$$LS = L * S$$

Slope Length Factor (L): Represents the effect of slope length on erosion and is calculated as follows:

$$Slope\ Length\ Factor\ (L) = (\lambda/72.6)^{m}$$

Where:

ϑ = slope angle (in radians)
λ: the horizontal projection in feet (not distance parallel to the soil surface)

Ratio of Rill/Interrill Erosion (β) = $(\sin\vartheta/0.0896)/[3.0* (\sin\vartheta)^{0.8} + 0.56]$

Slope Length Exponent (m) = β / (1+β)

Slope Steepness Factor (S): Represents the effect of slope steepness on erosion and is calculated as follows:

$$Slope\ Steepness\ Factor\ (S) = 10.8 * \sin\theta + 0.03 \qquad s < 9\%$$

$$= 16.8 * \sin\theta - 0.50 \qquad s \geq 9\%$$

s = slope steepness (%)

For ease of calculation, the attached RUSLE Calculation Sheet was developed to compute the Topographic Factor "LS" based on input values. **This represents a 3:1 (Horizontal:Vertical) slope for final cover grades, with storm water benches spaced approximately every 30 vertical feet, 90 horizontal feet. A 30V:90H spacing was used as a maximum for calculation purposes. More typical space is 25V:75H. The calculation results in an "LS" factor of 5.55.**

Crop Management Factor (C): The Crop Management "C" factor represents the effect of the land use, like plants, soil cover, soil biomass, and soil disturbing activities on soil erosion. Its value is an average soil loss ratio weighted according to the distribution of "R" during the year, and is influenced by canopy, ground cover, soil surface roughness, and prior land use. The "C" factor can be determined using the following table:

Vegetal Canopy			Cover That Contacts the Surface					
Type and Height Of Raised Canopy [2]	Canopy Cover [3]	Type [4]	0	20	40	60	80	95-100
Column No.:	2	3	4	5	6	7	8	9
No appreciable Canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.090	.043	.011
Canopy of tall Weeds or short Brush (0.5 m fall ht.)	25	G	.36	.17	.09	.038	.012	.003
		W	.36	.20	.13	.082	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.075	.039	.011
	75	G	.17	.10	.06	.031	.011	.003
		W	.17	.12	.09	.067	.038	.011
Appreciable brush Or bushes (2 m fall ht.)	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.085	.042	.011
	50	G	.34	.16	.085	.038	.012	.003
		W	.34	.19	.13	.081	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.077	.041	.011
Trees but no Appreciable low Brush (4 m Fall ht.)	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.087	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.085	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.083	.041	.011

1 The listed C values assume that the vegetation and mulch are randomly distributed over the entire area.
2 Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to drop fall height and is negligible if fall height exceeds 33-ft.
3 Portion of total-area surface that would be hidden from view by canopy in a vertical projection, (a bird's-eye view).
4 G: Cover at surface is grass, grass like plants, decaying compacted duff, or litter at least 2 inches deep.
W: Cover at surface is mostly broadleaf herbaceous plant (as weeds with little lateral-root network near the surface) or undecayed residues or both

Figure 2. "C" for permanent pasture, range and idle land (Taken from Reference 4, Table 10)

A Crop Management "C" factor of 0.013 was chosen assuming 80 percent ground cover with no appreciable canopy.

Support Practice Factor (P): The "P" factor represents the impact of supports on erosion rates and is the ratio of soil loss from an area with supporting practices in place to that from an identical area without any supporting practices. It typically affects erosion by redirecting runoff around the slope to reduce runoff erosivity or by slowing down the runoff to cause deposition. "P" values range between 0 and 1.0. **A Support Practice "P" value of 1.0 was chosen to provide a conservative estimate of erosion from the site.**

III. CONCLUSIONS

As shown in the attached RUSLE Calculation sheet, the proposed landfill will result in a predicted soil loss of **4.98 tons per acre per year**. Therefore, the proposed maximum 33% slope and maximum 90-foot slope length provides an erosion rate less than the acceptable allowable soil loss of 5 tons per acre per year.

IV. REFERENCES

1. Revised Universal Soil Loss Equation Version 2 (RUSLE2) Draft User's Reference Guide, United States Department of Agriculture.
2. Stewart, B. A.,et al. Control of Water Pollution from Cropland: Vol. 1, A Manual for Guideline Development; Vol. 2, An Overview. ARS-H-5-1 and ARS-H-5-2 United States Department of Agriculture.
3. Predicting Soil Erosion by Water: A Guide to Conservative Planning With the Revised Universal Soil Loss Equation (RUSLE), Agricultural Handbook No. 703, United States Department of Agriculture.
4. Predicting Rainfall Erosion Losses, Agricultural Handbook 537, United States Department of Agriculture.

ATTACHMENT A

Revised Universal Soil Loss Calculation Sheet

Revised Universal Soil Loss Equation (RUSLE) Calculation

Scepter East Phase Landfill

$$E = R * K * LS * C * P$$

Rainfall Energy Factor	**R =**	**300**	(Waverly, Tennessee)
Soil Erodibility Factor	**K =**	**0.230**	(Silty Clay)
Horizontal Slope Length	**λ =**	**90**	ft
Slope Run	**H =**	**3**	ft
Slope Rise	**V =**	**1**	ft
Slope %	s =	33.333	%
Slope Angle	θ =	0.322	radians
	θ =	18.444	degree
Ratio of Rill/Interrilll Erosion	β =	(sin θ/0.0896)/[3.0(sin θ)^0.8 + 0.56]	
	β =	2.012	
Slope Length Exponent	m =	β/(1 + β)	
	m =	0.668	
Slope Length Factor	L =	(λ/72.6)^m	
	L =	1.154	
Slope Steepness Factor	S =	10.8 sin θ + 0.03	(s < 9%)
		16.8 sin θ - 0.50	(s > 9%)
	S =	4.813	
Topographic Factor	**LS =**	**L * S**	
	LS =	**5.555**	
Crop Management Factor	**C =**	**0.013**	
Support Practice Factor	**P =**	**1**	
Computed Soil Loss	**E =**	**R * K * LS * C * P**	

Computed Soil Loss	**E =**	**4.98**	**Tons / Acre / Year**

APPENDIX F.1G

Outlet Protection Design (Rev 2)

I. PURPOSE

The purpose of this analysis is to design the proposed riprap aprons for the Scepter Class II Disposal Facility in Waverly, TN consistent with state regulations.

II. SITE AND PROJECT DESCRIPTION

The riprap apron design was performed as part of the Part II Solid Waste Permit Application. The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management. Refer to Exhibit 1 for a depiction of the riprap apron locations in relation to the landfill.

The following sections summarize the design criteria, procedure, assumptions, and results of the channels design.

III. REGULATORY REQUIREMENTS / DESIGN CRITERIA

The below parts of the TDEC regulations specify requirements for the design of the channels:

Rule 0400-11-01-.04(2)(i)1.

The operator must design, construct, operate, and maintain a run-on control system capable of preventing flow onto the active portion of the facility for all flow up to and including peak discharge from a 24-hour, 25-year storm.

The landfill perimeter follows the sites existing high point ridge line, therefore run-on flows are absent.

Rule 0400-11-01-.04(2)(i)2.

The operator must design, construct, operate, and maintain a run-off management system to collect and control at least the peak flow resulting from a 24-hour, 25 year storm.

The results of the analysis presented herein show that the riprap aprons are designed to collect and control at least the peak flow resulting from a 25-year/24-hour storm.

Rule 0400-11-01-.04(2)(i)6.

The operator must take other erosion control measure (e.g., temporary mulching or seeding, silt barriers) as necessary to control erosion of the site.

Proper rip rap stone size and apron dimensions will be selected based on anticipated site characteristics (maximum discharge, culvert diameter, and tailwater depth) such that the designed rip rap aprons will reduce erosion all culvert discharge points.

IV. PROCEDURE

Design of the landfill site stormwater features was an iterative process beginning with basic assumptions and a proposed grading plan for the site. The hydraulic features of the channels were initially assumed and then confirmed through multiple iterations.

The AutoCAD Civil 3D software package was used to generate the proposed site grading plan and subsequently to determine drainage areas, volumes, and other site geometry. HydroCAD (version 10.00) modeling software was used to conduct the hydrologic calculations. The HydroCAD model was used to generate peak flow rates for the upstream watershed features. The Federal Highway Administration's Hydraulic Engineering Circular No. 14, Third Edition (HEC-14) was used to conduct hydraulic design calculations of energy dissipators for culverts in this analysis with inputs based on maximum discharge, culvert diameter, tailwater depth, and other design assumptions.

The conducted riprap apron design was analyzed once using the maximum culvert discharge anticipated onsite for the 24-hour, 25-year storm event. All other variables remain constant between all culvert discharge points. Therefore, the critical discharge point was analyzed and applied to all locations.

V. NOTES/ASSUMPTIONS

The following is a list of key notes and assumptions made in completing this analysis.

- See F1.B for culvert maximum discharges and culvert sizing.
- Tennessee Department of Transportation Riprap Sizes were used
- Tailwater was assumed to be 40% of the pipe diameter

VI. SUMMARY OF RESULTS

The proposed riprap aprons properly dissipate energy from flows generated by the 24-hour, 25-year with the follow minimum requirements (see Attachment A for detailed calculations):

Apron Length = 12.5 feet

Apron Depth = 1.8 feet

Apron Width (at apron end) = 15.8 feet

Apron Width (at apron beginning) = 7.5 feet

Rip Rap Size = 8.4 inch (equivalent TDOT A-1 Riprap)

VII. CONCLUSIONS

The proposed riprap aprons properly dissipate energy from the flows generated by the 24-hour, 25-year storm as stipulated by TDEC regulations. Calculations for proper conveyance of upstream and downstream flows were not presented in this analysis and will be performed separately. Refer to accompanying calculations for upstream and downstream conveyance design features (storm water terraces and letdowns, open channels, culverts, and sediment basins) conveyance features.

VIII. ATTACHMENTS

Figures:
Exhibit 1: Drainage Map

Attachments:
Attachment A: Riprap Apron Design Calculations

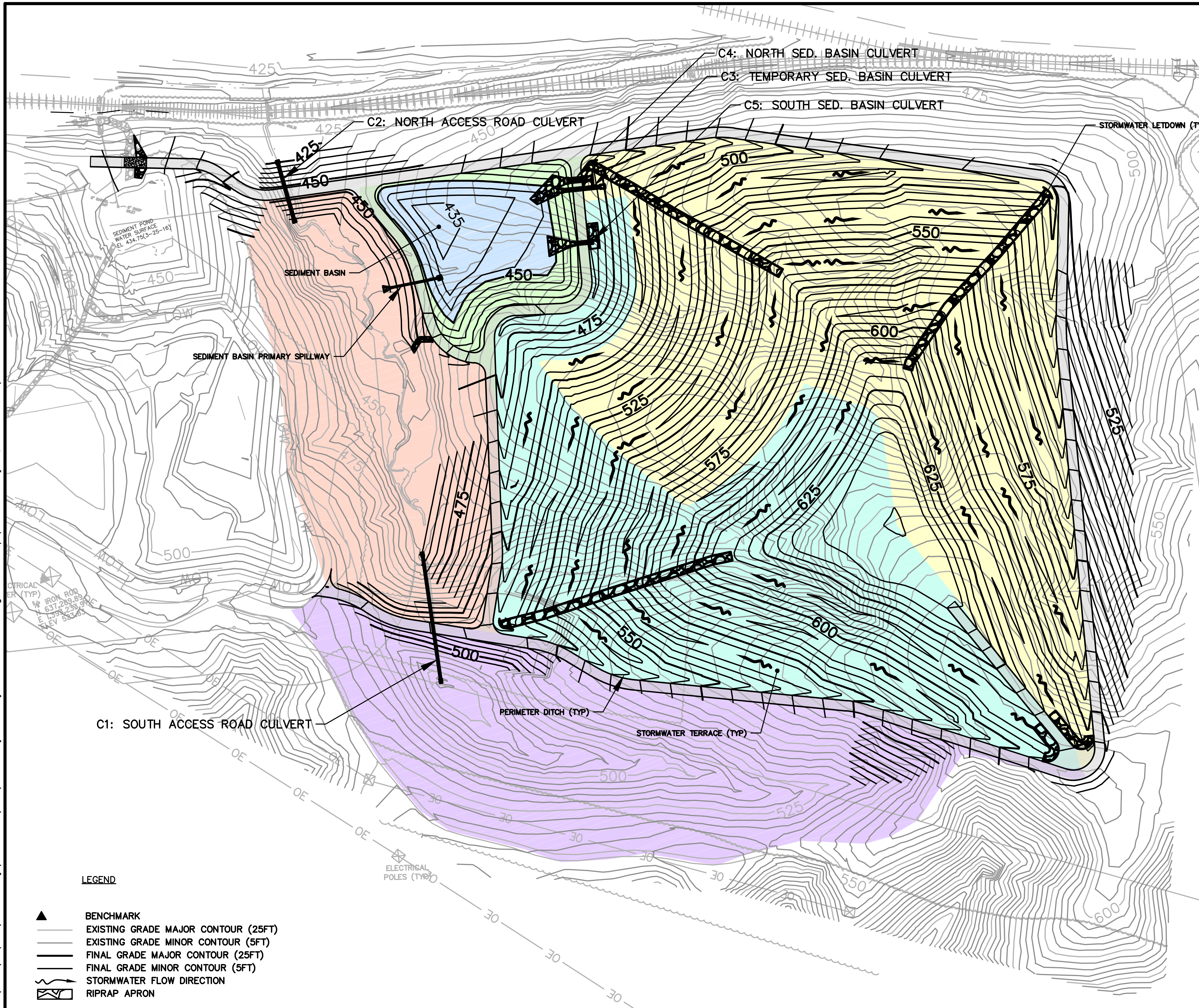
IX. REFERENCES

1. TDEC, *"Rules of Tennessee Department of Environment and Conservation, Chapter 0400-11-01 – Solid Waste Processing and Disposal"*, Solid Waste Management, September 2012.

2. U.S. Department of Transportation Federal Highway Administration, *"Hydraulic Engineering Circular No. 14, Third Edition"*, Hydraulic Design of Energy Dissipaters for Culverts and Channels, July 2006.

EXHIBIT 1

Drainage Map



NORTH - NAD83

0 200' 400'

GRAPHIC SCALE

DRAINAGE AREAS

NORTH SED. BASIN CULVERT AREA = 15.99 ACRES CN = 78 TC = 14.0 MINS
SOUTH SED. BASIN CULVERT AREA = 12.06 ACRES CN = 78 TC = 16.5 MINS
SED. BASIN OPEN WATER AREA = 1.50 ACRES CN = 98 TC = 5.0 MINS
SED. BASIN SOIL AREA = 1.50 ACRES CN = 78 TC = 5.0 MINS
NORTH ACCESS ROAD CULVERT AREA = 7.260 CN = 49 TC = 6.0 MINS
SOUTH ACCESS ROAD CULVERT AREA-1 = 9.24 ACRES; CN-1 = 49 AREA-2 = 1 ACRE; CN-2 = 77 TC = 9.9 MINS

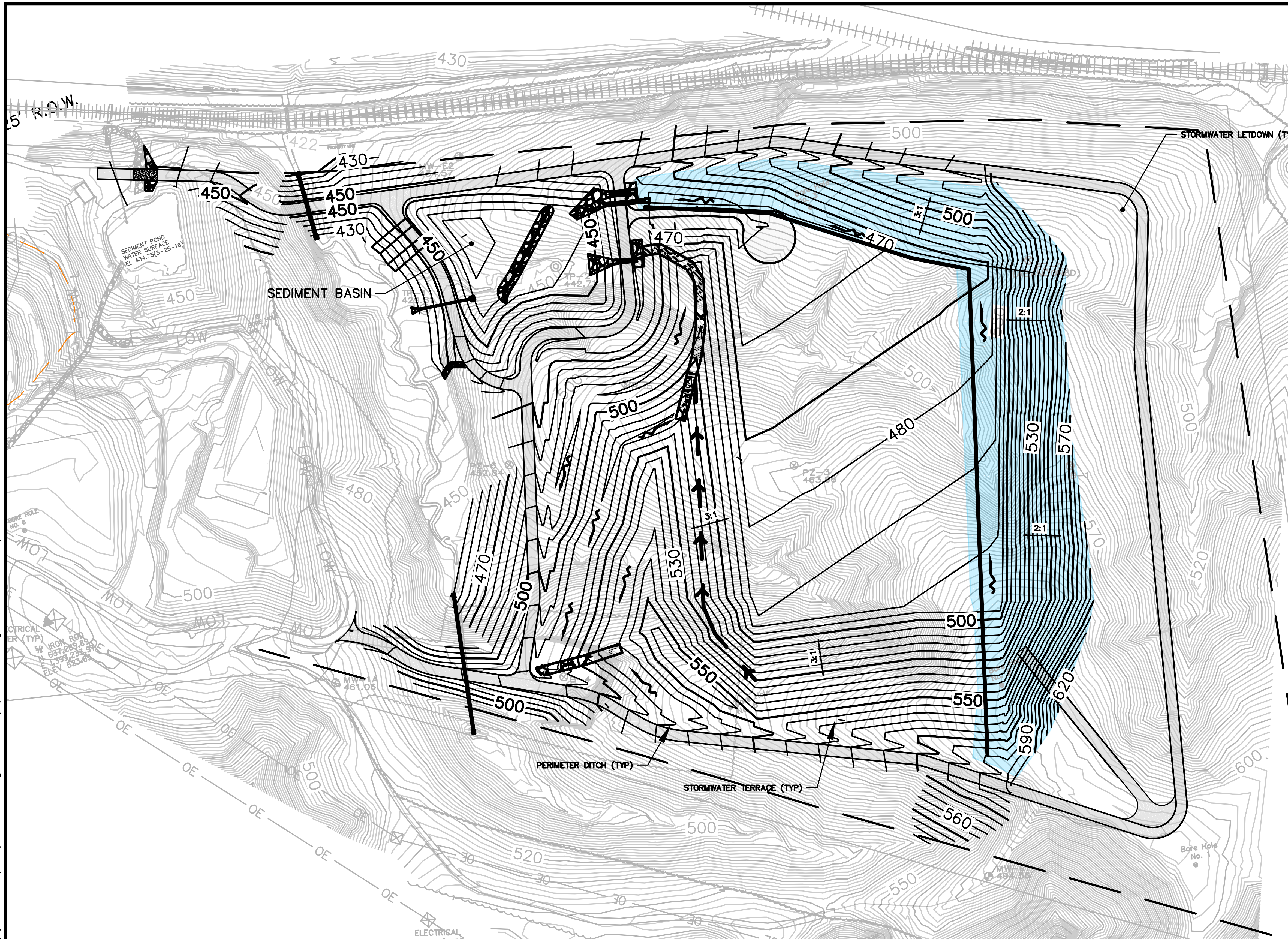
AECOM

SCEPTER, INC.

WAVERLY, TENNESSEE

ROAD CULVERT DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/4/17	1A



0 100' 200'

GRAPHIC SCALE

DRAINAGE AREAS

TEMPORARY SED. BASIN CULVERT
AREA = 6.51 ACRES
CN = 78
TC = 7.6 MINS

LEGEND

- ▲ BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (25FT)
- EXISTING GRADE MINOR CONTOUR (5FT)
- FINAL GRADE MAJOR CONTOUR (25FT)
- FINAL GRADE MINOR CONTOUR (5FT)
- STORMWATER FLOW DIRECTION

AECOM

SCEPTER, INC.
WAVERLY, TENNESSEE

TEMPORARY SEDIMENT BASIN CULVERT DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/4/17	1B

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ATTACHMENT A

Riprap Apron Design Calculations

Reference: HEC-14 Hydraulic Design of Energy Dissipators for Culverts and Channels Chapter 10

① Determine Riprap Apron Riprap Size

$$D_{50} = 0.2D\left(\frac{Q}{\sqrt{g}\,D^{2.5}}\right)^{4/3}\left(\frac{D}{TW}\right)$$

where D_{50} = rip rap size, ft
Q = design discharge = Q_{max} = 29.18 cfs
↳ Individual maximum culvert discharge for all riprap aprons within the East Phase Permit submittal. Q_{max} = 58.35 cfs / 2 = 29.18 cfs
D = Culvert diameter (circular), ft = 2.5 feet
TW = tailwater depth, ft = 0.4D = 0.4(2.5) = 1 foot
g = acceleration of gravity = 32.2 ft/s^2

$$D_{50} = 0.2(2.5\text{ ft})\left(\frac{29.18\text{ cfs}}{\sqrt{32.2\text{ ft/s}^2}\,(2.5)^{2.5}}\right)^{4/3}\left(\frac{2.5\text{ ft}}{1\text{ ft}}\right)$$

= 0.52 feet

D_{50} = 6.3 inches → use TDOT A-1 Riprap, D_{50} = 9 inches

② Determine Riprap Class and Min. Apron Dimensions

Table 10.1. Example Riprap Classes and Apron Dimensions

Class	D_{50} (mm)	D_{50} (in)	Apron Length[1]	Apron Depth
1	125	5	4D	$3.5D_{50}$
2	150	6	4D	$3.3D_{50}$
3	250	10	5D	$2.4D_{50}$
4	350	14	6D	$2.2D_{50}$
5	500	20	7D	$2.0D_{50}$
6	550	22	8D	$2.0D_{50}$

[1]D is the culvert rise.

Apron Length = 5D = 5(2.5 ft)
= 12.5 feet (min)

Apron Depth = $2.4D_{50}$ = 2.4(9 in)
= 21.6 in
= 1.8 feet (min)

Apron Width (at apron end) = 3D + 2/3 L
= 3(2.5 ft) + 2/3(12.5 ft)
= 15.8 ft (min)

Apron Width (at apron beginning) = 3D
= 3(2.5 ft)
= 7.5 ft (min)

APPENDIX F.1H

Exterior Daylight Channel Design

I. PURPOSE

The purpose of this analysis is to design the proposed exterior daylight channels for the Scepter Class II Disposal Facility in Waverly, TN consistent with state regulations. These channels are created when daylighting the proposed landfill grades onto existing valleys grades.

II. SITE AND PROJECT DESCRIPTION

The channel design was performed as part of the Part II Solid Waste Permit Application. The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management. Refer to Exhibit 1 for a depiction of the channel locations in relation to the landfill.

The following sections summarize the design criteria, procedure, assumptions, and results of the channels design.

III. REGULATORY REQUIREMENTS / DESIGN CRITERIA

The below parts of the TDEC regulations specify requirements for the design of the channels:

Rule 0400-11-01-.04(2)(i)1.

The operator must design, construct, operate, and maintain a run-on control system capable of preventing flow onto the active portion of the facility for all flow up to and including peak discharge from a 24-hour, 25-year storm.

Rule 0400-11-01-.04(2)(i)2.

The operator must design, construct, operate, and maintain a run-off management system to collect and control at least the peak flow resulting from a 24-hour, 25 year storm.

The landfill perimeter follows the sites existing high point ridge line, therefore run-on flows are absent. However, daylighting proposed grades reestablish existing valley floors. These reestablished valleys are designed in accordance with the TDEC Solid Waste requirements and properly collect and control at least the peak flow resulting from a 25-year/24-hour storm.

Run-off management is cover under accompanying calculations; reference Appendix F.

Rule 0400-11-01-.04(2)(i)6.

The operator must take other erosion control measure (e.g., temporary mulching or seeding, silt barriers) as necessary to control erosion of the site.

Proper channel lining will be selected based on anticipated velocities such that erosion within the channels will be significantly reduced.

IV. PROCEDURE

Design of the landfill site stormwater features was an iterative process beginning with basic assumptions and a proposed grading plan for the site. The hydraulic features of the channels were initially assumed and then confirmed through multiple iterations.

The AutoCAD Civil 3D software package was used to generate the proposed site grading plan and subsequently to determine drainage areas, volumes, and other site geometry. HydroCAD (version 10.00) modeling software was used to conduct the hydrologic calculations. The Federal Highway Administration's Hydraulic Toolbox was used to conduct hydraulic calculations for this analysis with inputs based on the site geometry, rainfall data, and other design assumptions.

The HydroCAD model was used to generate peak flow rates for the upstream watershed features. Hydraulic Toolbox was used to generate channel velocities and water surface elevations at the most critical points along the channel alignment. The proposed channels will pass flows equal to and lesser than those generated by the 24-hour, 25-year storm.

V. NOTES/ASSUMPTIONS

The following is a list of key notes and assumptions made in completing this analysis.

- A design parameter minimum six (6) inches of freeboard was used.
- Within the HydroCAD program, the runoff was calculated using the SCS TR-20 method.
- Runoff curve numbers (CN) used in the analysis were as follows:
    - 78 for landfill vegetated final cap cover and offsite areas,
    - 50 for undisturbed forested areas,
    - 77 for exposed and disturbed soil (daily cover); however exposed and disturbed soil was ignored to be conservative and 78 was used within landfill construction footprint.
- The time of concentration was calculated using the Curve Number Method in HydroCAD which takes inputs for each drainage area of the longest hydraulic flow path and average land slope.

VI. SUMMARY OF RESULTS

The channels properly control and convey the water volume generated by the 24-hour, 25-year storm such that the peak water surface elevation stays safely below the impounding sideslopes with greater than six (6) inches of freeboard. A rip rap channel lining was chosen for all channels as the shear stress is under 3.6 lb/ft^2. TRM maybe used as a replacement to riprap where deemed necessary at the time of final design. See Table 1 for further detail.

Table 1 – Exterior Daylight Channel Summary

Channel ID	Peak Discharge (cfs)	Slope (ft/ft)	Flow Depth (feet)	Velocity (fps)	Shear Stress (lb/ft^2)
South	3.60	0.26	0.383	7.006	2.987
East	25.26	0.12	0.978	8.811	3.472

VII. CONCLUSIONS

The proposed grading of the landfill site in combination with the design of the channels as presented above is sufficient to safely control and convey the 24-hour, 25-year storm as stipulated by TDEC regulations.

VIII. ATTACHMENTS

Figures:
Exhibit 1: Drainage Map

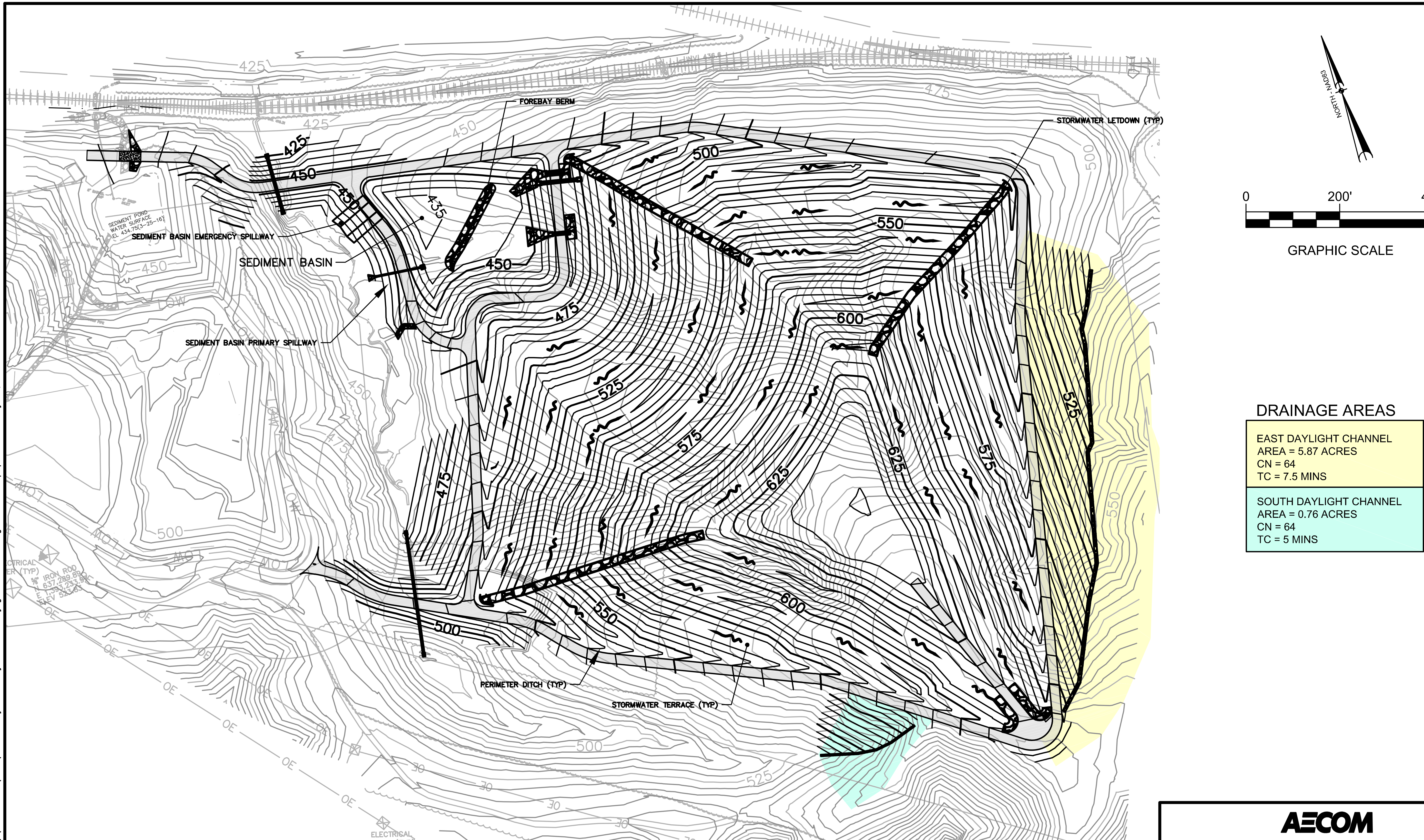
Attachments:
Attachment A: NOAA Precipitation Frequency Data
Attachment B: Perimeter Ditch: Drainage Map, HydroCAD report, and Hydraulic Toolbox Report
Attachment C: Permissible Shear Stress

IX. REFERENCES

1. TDEC, *"Rules of Tennessee Department of Environment and Conservation, Chapter 0400-11-01 – Solid Waste Processing and Disposal"*, Solid Waste Management, September 2012.

EXHIBIT 1

Drainage Map



DRAINAGE AREAS

EAST DAYLIGHT CHANNEL	AREA = 5.87 ACRES CN = 64 TC = 7.5 MINS
SOUTH DAYLIGHT CHANNEL	AREA = 0.76 ACRES CN = 64 TC = 5 MINS

LEGEND

- BENCHMARK
- EXISTING GRADE MAJOR CONTOUR (25FT)
- EXISTING GRADE MINOR CONTOUR (5FT)
- FINAL GRADE MAJOR CONTOUR (25FT)
- FINAL GRADE MINOR CONTOUR (5FT)
- STORMWATER FLOW DIRECTION

AECOM

SCEPTER, INC.

WAVERLY, TENNESSEE

EXTERIOR DAYLIGHT CHANNELS DRAINAGE MAP

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NP	MM	60398526	5/16/17	1

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ATTACHMENT A

NOAA Precipitation Frequency Data

NOAA Atlas 14, Volume 2, Version 3
Location name: Waverly, Tennessee, US*
Latitude: 36.0689°, Longitude: -87.9462°
Elevation: 523 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.405 (0.368-0.450)	0.476 (0.434-0.529)	0.547 (0.497-0.607)	0.604 (0.548-0.668)	0.675 (0.610-0.746)	0.728 (0.655-0.803)	0.781 (0.699-0.862)	0.832 (0.742-0.917)	0.898 (0.794-0.990)	0.948 (0.833-1.05)
10-min	0.647 (0.588-0.718)	0.761 (0.694-0.845)	0.876 (0.796-0.972)	0.965 (0.877-1.07)	1.08 (0.972-1.19)	1.16 (1.04-1.28)	1.24 (1.11-1.37)	1.32 (1.18-1.45)	1.42 (1.26-1.57)	1.49 (1.31-1.65)
15-min	0.809 (0.735-0.898)	0.957 (0.872-1.06)	1.11 (1.01-1.23)	1.22 (1.11-1.35)	1.36 (1.23-1.51)	1.47 (1.32-1.62)	1.57 (1.40-1.73)	1.66 (1.48-1.83)	1.79 (1.58-1.97)	1.88 (1.65-2.07)
30-min	1.11 (1.01-1.23)	1.32 (1.20-1.47)	1.58 (1.43-1.75)	1.77 (1.61-1.96)	2.02 (1.83-2.23)	2.21 (1.99-2.44)	2.40 (2.15-2.65)	2.59 (2.31-2.86)	2.84 (2.51-3.14)	3.04 (2.67-3.35)
60-min	1.38 (1.26-1.54)	1.66 (1.51-1.84)	2.02 (1.83-2.24)	2.30 (2.09-2.55)	2.69 (2.43-2.97)	2.99 (2.70-3.31)	3.31 (2.96-3.65)	3.63 (3.24-4.01)	4.08 (3.61-4.50)	4.43 (3.89-4.89)
2-hr	1.59 (1.44-1.76)	1.91 (1.73-2.11)	2.33 (2.11-2.59)	2.69 (2.43-2.98)	3.19 (2.87-3.52)	3.60 (3.22-3.98)	4.03 (3.59-4.46)	4.49 (3.98-4.97)	5.14 (4.52-5.70)	5.67 (4.95-6.29)
3-hr	1.72 (1.57-1.89)	2.05 (1.87-2.27)	2.52 (2.29-2.78)	2.91 (2.64-3.20)	3.45 (3.12-3.80)	3.91 (3.52-4.30)	4.39 (3.93-4.82)	4.90 (4.37-5.39)	5.64 (4.98-6.20)	6.24 (5.47-6.88)
6-hr	2.13 (1.93-2.35)	2.53 (2.30-2.80)	3.10 (2.82-3.43)	3.58 (3.25-3.95)	4.26 (3.84-4.69)	4.83 (4.33-5.32)	5.43 (4.85-5.98)	6.09 (5.40-6.70)	7.01 (6.17-7.72)	7.78 (6.78-8.58)
12-hr	2.58 (2.36-2.85)	3.08 (2.82-3.40)	3.79 (3.46-4.18)	4.38 (3.99-4.82)	5.22 (4.73-5.74)	5.92 (5.34-6.51)	6.66 (5.97-7.32)	7.46 (6.65-8.20)	8.60 (7.59-9.45)	9.54 (8.36-10.5)
24-hr	3.14 (2.88-3.44)	3.76 (3.45-4.12)	4.64 (4.25-5.09)	5.37 (4.91-5.87)	6.39 (5.82-6.98)	7.23 (6.56-7.89)	8.12 (7.32-8.85)	9.06 (8.12-9.88)	10.4 (9.22-11.3)	11.5 (10.1-12.5)
2-day	3.75 (3.46-4.10)	4.49 (4.14-4.92)	5.53 (5.09-6.04)	6.36 (5.84-6.93)	7.52 (6.89-8.19)	8.46 (7.72-9.21)	9.45 (8.56-10.3)	10.5 (9.44-11.4)	11.9 (10.6-13.0)	13.1 (11.6-14.3)
3-day	4.01 (3.69-4.37)	4.80 (4.42-5.24)	5.89 (5.43-6.43)	6.77 (6.22-7.37)	7.98 (7.31-8.69)	8.96 (8.17-9.75)	9.97 (9.04-10.9)	11.0 (9.94-12.0)	12.5 (11.2-13.7)	13.6 (12.1-15.0)
4-day	4.26 (3.93-4.64)	5.11 (4.71-5.56)	6.26 (5.77-6.82)	7.18 (6.61-7.81)	8.45 (7.74-9.18)	9.46 (8.62-10.3)	10.5 (9.52-11.4)	11.6 (10.4-12.6)	13.0 (11.7-14.3)	14.2 (12.6-15.6)
7-day	5.06 (4.65-5.50)	6.06 (5.58-6.59)	7.43 (6.83-8.07)	8.52 (7.81-9.25)	10.0 (9.17-10.9)	11.3 (10.2-12.2)	12.6 (11.4-13.6)	13.9 (12.5-15.1)	15.7 (14.0-17.2)	17.2 (15.2-18.8)
10-day	5.77 (5.34-6.22)	6.90 (6.38-7.45)	8.39 (7.75-9.05)	9.55 (8.81-10.3)	11.1 (10.2-12.0)	12.4 (11.3-13.3)	13.6 (12.4-14.7)	14.9 (13.5-16.1)	16.6 (15.0-18.0)	18.0 (16.1-19.6)
20-day	7.88 (7.35-8.46)	9.36 (8.74-10.1)	11.1 (10.4-12.0)	12.5 (11.6-13.4)	14.2 (13.2-15.3)	15.5 (14.4-16.7)	16.8 (15.5-18.1)	18.1 (16.6-19.5)	19.7 (18.0-21.2)	20.9 (19.0-22.6)
30-day	9.63 (9.00-10.3)	11.4 (10.7-12.2)	13.4 (12.6-14.4)	15.0 (14.0-16.0)	16.9 (15.8-18.1)	18.4 (17.1-19.7)	19.9 (18.5-21.3)	21.4 (19.7-22.9)	23.2 (21.3-24.9)	24.6 (22.5-26.4)
45-day	12.1 (11.3-12.9)	14.3 (13.4-15.2)	16.7 (15.7-17.8)	18.5 (17.3-19.7)	20.8 (19.4-22.1)	22.4 (20.9-23.9)	24.1 (22.4-25.6)	25.6 (23.7-27.3)	27.6 (25.4-29.5)	29.0 (26.6-31.0)
60-day	14.5 (13.6-15.4)	17.1 (16.0-18.2)	19.8 (18.6-21.1)	21.8 (20.4-23.2)	24.3 (22.7-25.9)	26.1 (24.4-27.9)	27.8 (25.9-29.7)	29.5 (27.4-31.5)	31.4 (29.1-33.6)	32.9 (30.3-35.2)

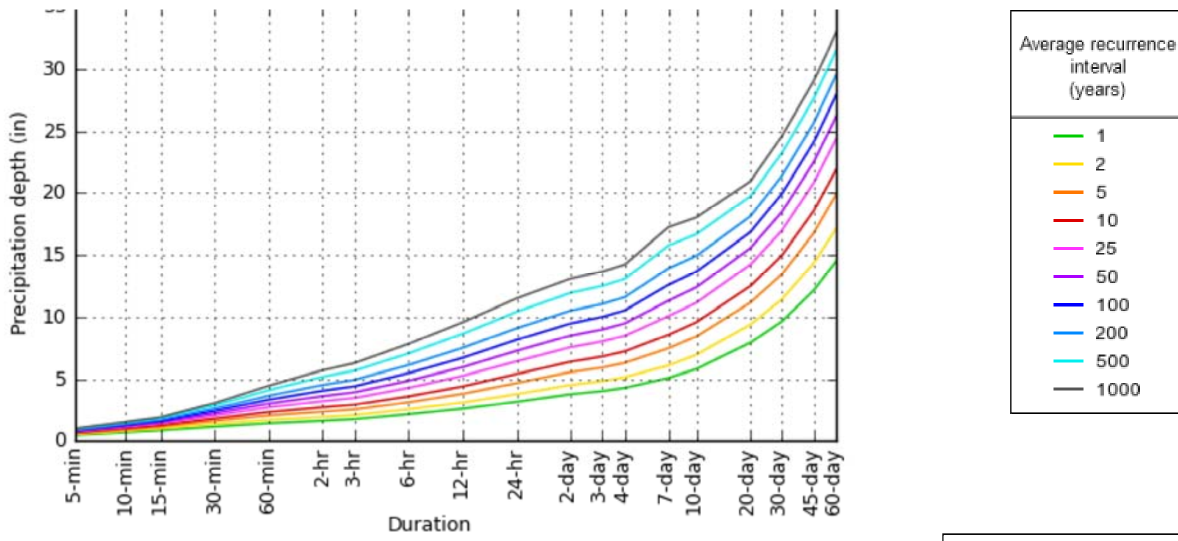
[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

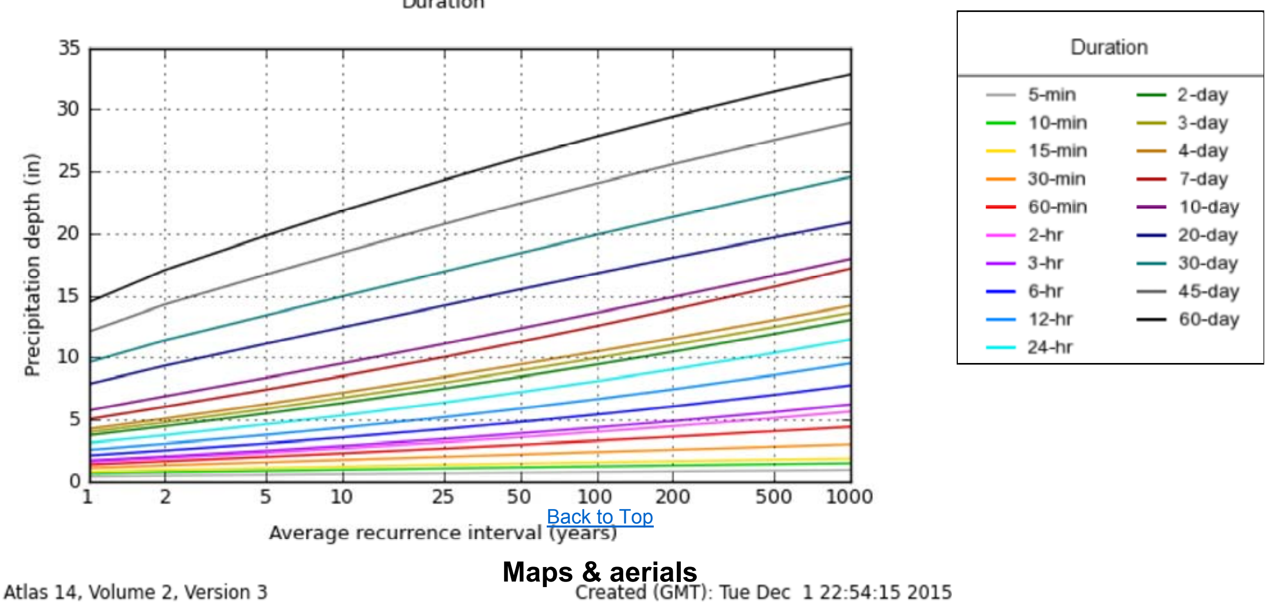
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical



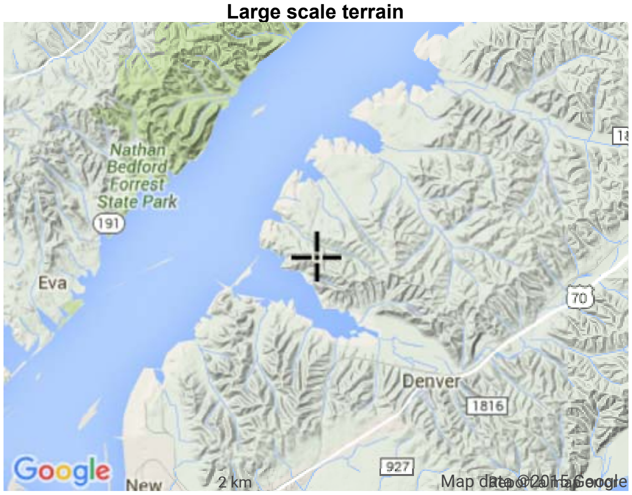


[Back to Top](#)

Maps & aerials

Small scale terrain

Large scale terrain



Large scale map

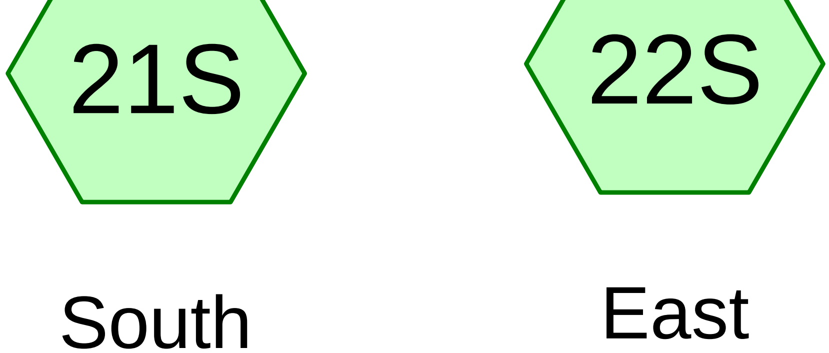
Large scale aerial

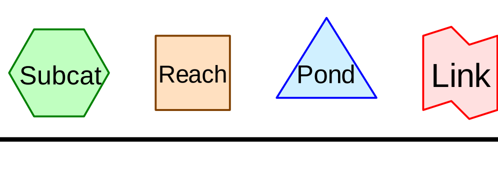
[Back to Top]

US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910

ATTACHMENT B

Perimeter Ditch Design





Routing Diagram for Exterior Daylight Channels
Prepared by AECOM, Printed 5/23/2017
HydroCAD® 10.00 s/n 01089 © 2013 HydroCAD Software Solutions LLC

Summary for Subcatchment 21S: South

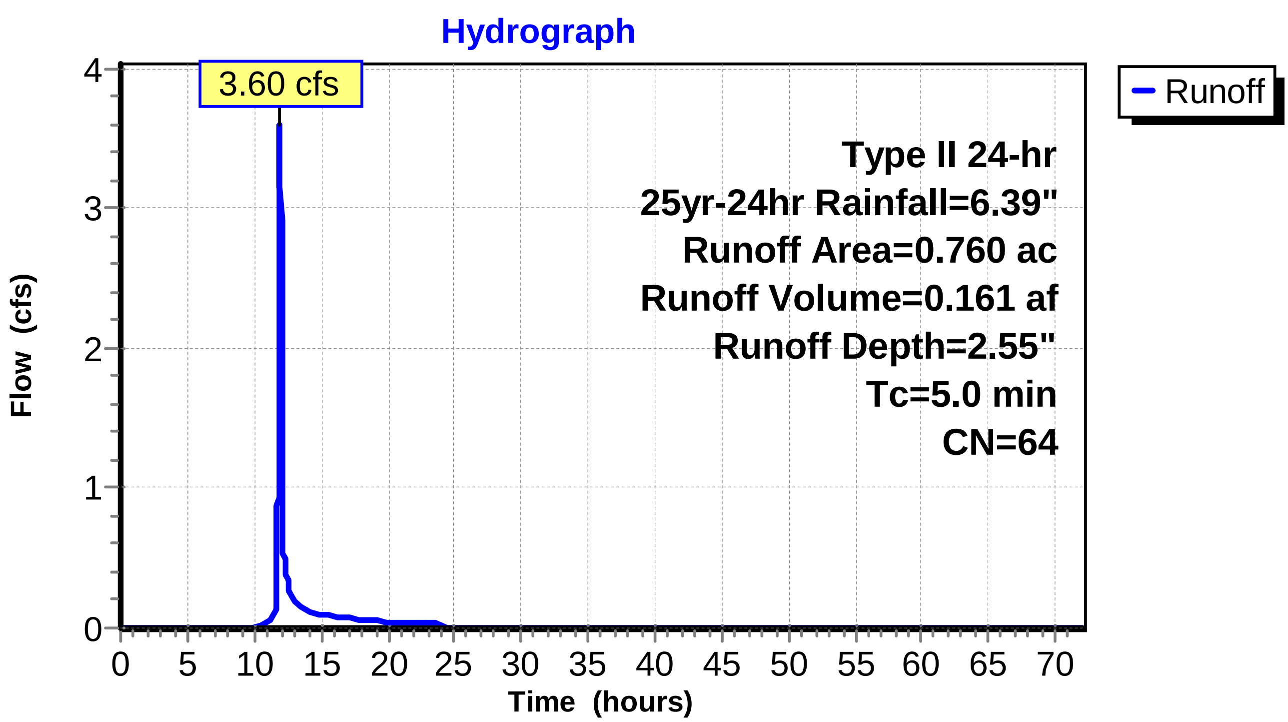
Runoff = 3.60 cfs @ 11.97 hrs, Volume= 0.161 af, Depth= 2.55"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	0.380	78	
*	0.380	50	wooded min cn
	0.760	64	Weighted Average
	0.760		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
5.0					**Direct Entry,**

Subcatchment 21S: South



Hydrograph for Subcatchment 21S: South

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.00
8.00	0.77	0.00	0.00
9.00	0.94	0.00	0.00
10.00	1.16	0.00	0.00
11.00	1.50	0.02	**0.04**
12.00	4.24	1.11	**3.16**
13.00	4.93	1.54	0.20
14.00	5.24	1.74	0.13
15.00	5.45	1.88	0.10
16.00	5.62	2.00	0.08
17.00	5.76	2.10	0.07
18.00	5.89	2.18	0.06
19.00	5.99	2.26	0.05
20.00	6.08	2.32	0.05
21.00	6.16	2.38	0.04
22.00	6.24	2.44	0.04
23.00	6.32	2.49	0.04
24.00	**6.39**	**2.55**	0.04
25.00	6.39	2.55	0.00
26.00	6.39	2.55	0.00
27.00	6.39	2.55	0.00
28.00	6.39	2.55	0.00
29.00	6.39	2.55	0.00
30.00	6.39	2.55	0.00
31.00	6.39	2.55	0.00
32.00	6.39	2.55	0.00
33.00	6.39	2.55	0.00
34.00	6.39	2.55	0.00
35.00	6.39	2.55	0.00
36.00	6.39	2.55	0.00
37.00	6.39	2.55	0.00
38.00	6.39	2.55	0.00
39.00	6.39	2.55	0.00
40.00	6.39	2.55	0.00
41.00	6.39	2.55	0.00
42.00	6.39	2.55	0.00
43.00	6.39	2.55	0.00
44.00	6.39	2.55	0.00
45.00	6.39	2.55	0.00
46.00	6.39	2.55	0.00
47.00	6.39	2.55	0.00
48.00	6.39	2.55	0.00
49.00	6.39	2.55	0.00
50.00	6.39	2.55	0.00
51.00	6.39	2.55	0.00
52.00	6.39	2.55	0.00
53.00	6.39	2.55	0.00
54.00	6.39	2.55	0.00
55.00	6.39	2.55	0.00
56.00	6.39	2.55	0.00
57.00	6.39	2.55	0.00
58.00	6.39	2.55	0.00
59.00	6.39	2.55	0.00
60.00	6.39	2.55	0.00
61.00	6.39	2.55	0.00
62.00	6.39	2.55	0.00
63.00	6.39	2.55	0.00
64.00	6.39	2.55	0.00
65.00	6.39	2.55	0.00
66.00	6.39	2.55	0.00
67.00	6.39	2.55	0.00
68.00	6.39	2.55	0.00
69.00	6.39	2.55	0.00
70.00	6.39	2.55	0.00
71.00	6.39	2.55	0.00
72.00	6.39	2.55	0.00

Summary for Subcatchment 22S: East

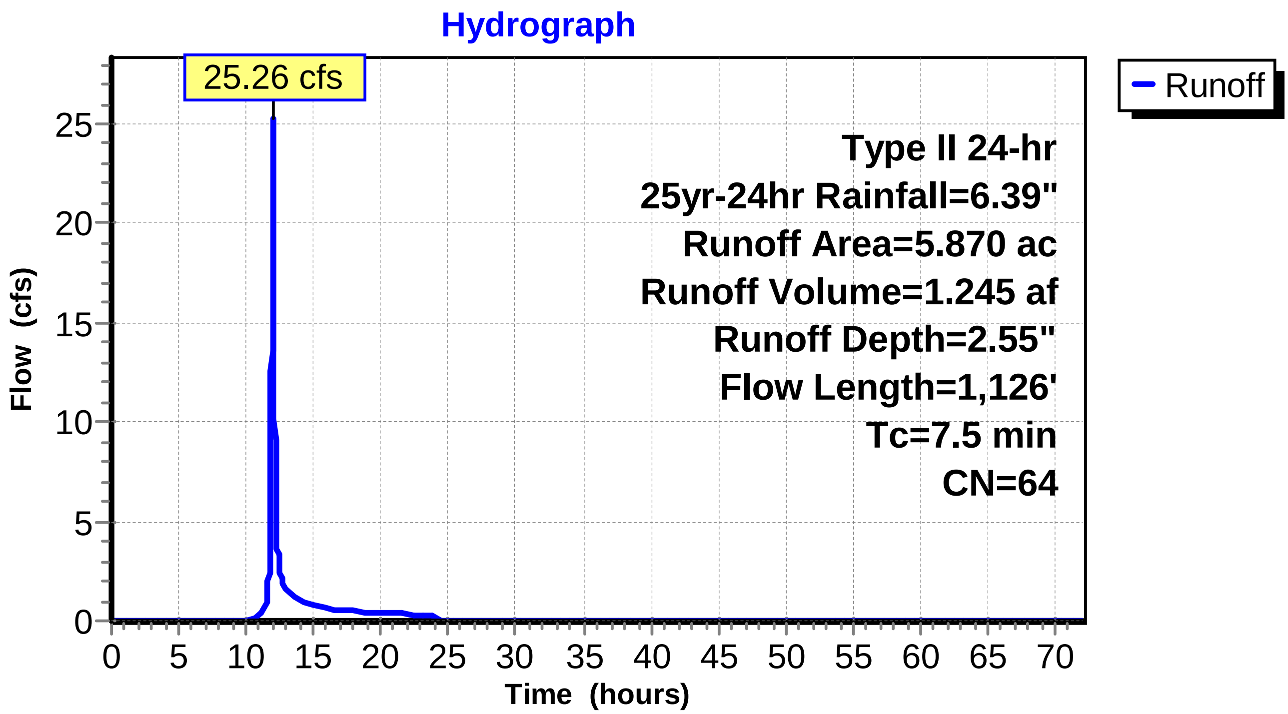
Runoff = 25.26 cfs @ 11.99 hrs, Volume= 1.245 af, Depth= 2.55"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-72.00 hrs, dt= 0.01 hrs
Type II 24-hr 25yr-24hr Rainfall=6.39"

	Area (ac)	CN	Description
*	3.000	78	
*	2.870	50	wooded min cn
	5.870	64	Weighted Average
	5.870		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.6	42	0.0200	0.11		**Sheet Flow, Sheet Flow** Grass: Dense n= 0.240 P2= 3.76"
0.5	91	0.0350	2.81		**Shallow Concentrated Flow, Shallow Concentrated Flow** Grassed Waterway Kv= 15.0 fps
0.4	993	0.1212	40.14	164.55	**Channel Flow, Daylight Channel** Area= 4.1 sf Perim= 6.0' r= 0.68' n= 0.010 PVC, smooth interior
7.5	1,126	Total			

Subcatchment 22S: East



Hydrograph for Subcatchment 22S: East

Time (hours)	Precip. (inches)	Excess (inches)	Runoff (cfs)
0.00	0.00	0.00	0.00
1.00	0.07	0.00	0.00
2.00	0.14	0.00	0.00
3.00	0.22	0.00	0.00
4.00	0.31	0.00	0.00
5.00	0.40	0.00	0.00
6.00	0.51	0.00	0.00
7.00	0.63	0.00	0.00
8.00	0.77	0.00	0.00
9.00	0.94	0.00	0.00
10.00	1.16	0.00	0.00
11.00	1.50	0.02	**0.27**
12.00	4.24	1.11	**25.19**
13.00	4.93	1.54	1.62
14.00	5.24	1.74	0.99
15.00	5.45	1.88	0.79
16.00	5.62	2.00	0.62
17.00	5.76	2.10	0.55
18.00	5.89	2.18	0.49
19.00	5.99	2.26	0.43
20.00	6.08	2.32	0.36
21.00	6.16	2.38	0.34
22.00	6.24	2.44	0.33
23.00	6.32	2.49	0.32
24.00	**6.39**	**2.55**	0.31
25.00	6.39	2.55	0.00
26.00	6.39	2.55	0.00
27.00	6.39	2.55	0.00
28.00	6.39	2.55	0.00
29.00	6.39	2.55	0.00
30.00	6.39	2.55	0.00
31.00	6.39	2.55	0.00
32.00	6.39	2.55	0.00
33.00	6.39	2.55	0.00
34.00	6.39	2.55	0.00
35.00	6.39	2.55	0.00
36.00	6.39	2.55	0.00
37.00	6.39	2.55	0.00
38.00	6.39	2.55	0.00
39.00	6.39	2.55	0.00
40.00	6.39	2.55	0.00
41.00	6.39	2.55	0.00
42.00	6.39	2.55	0.00
43.00	6.39	2.55	0.00
44.00	6.39	2.55	0.00
45.00	6.39	2.55	0.00
46.00	6.39	2.55	0.00
47.00	6.39	2.55	0.00
48.00	6.39	2.55	0.00
49.00	6.39	2.55	0.00
50.00	6.39	2.55	0.00
51.00	6.39	2.55	0.00
52.00	6.39	2.55	0.00
53.00	6.39	2.55	0.00
54.00	6.39	2.55	0.00
55.00	6.39	2.55	0.00
56.00	6.39	2.55	0.00
57.00	6.39	2.55	0.00
58.00	6.39	2.55	0.00
59.00	6.39	2.55	0.00
60.00	6.39	2.55	0.00
61.00	6.39	2.55	0.00
62.00	6.39	2.55	0.00
63.00	6.39	2.55	0.00
64.00	6.39	2.55	0.00
65.00	6.39	2.55	0.00
66.00	6.39	2.55	0.00
67.00	6.39	2.55	0.00
68.00	6.39	2.55	0.00
69.00	6.39	2.55	0.00
70.00	6.39	2.55	0.00
71.00	6.39	2.55	0.00
72.00	6.39	2.55	0.00

Hydraulic Analysis Report

Project Data

Project Title:

Designer:

Project Date: Tuesday, May 16, 2017

Project Units: U.S. Customary Units

Notes:

Channel Analysis: South

Notes:

Input Parameters

Channel Type: Triangular

Side Slope 1 (Z1): 3.0000 ft/ft

Side Slope 2 (Z2): 4.0000 ft/ft

Longitudinal Slope: 0.2600 ft/ft

Manning's n: 0.0350

Flow: 3.6000 cfs

Result Parameters

Depth: 0.3832 ft

Area of Flow: 0.5139 ft^2

Wetted Perimeter: 2.7916 ft

Hydraulic Radius: 0.1841 ft

Average Velocity: 7.0056 ft/s

Top Width: 2.6822 ft

Froude Number: 2.8206

Critical Depth: 0.5825 ft

Critical Velocity: 3.0311 ft/s

Critical Slope: 0.0278 ft/ft

Critical Top Width: 4.16 ft

Calculated Max Shear Stress: 6.2166 lb/ft^2

Calculated Avg Shear Stress: 2.9865 lb/ft^2

Channel Analysis: East

Notes:

Input Parameters

Channel Type: Triangular

Side Slope 1 (Z1): 3.0000 ft/ft

Side Slope 2 (Z2): 3.0000 ft/ft

Longitudinal Slope: 0.1200 ft/ft

Manning's n: 0.0350

Flow: 25.2600 cfs

Result Parameters

Depth: 0.9776 ft

Area of Flow: 2.8668 ft^2

Wetted Perimeter: 6.1826 ft

Hydraulic Radius: 0.4637 ft

Average Velocity: 8.8111 ft/s

Top Width: 5.8653 ft

Froude Number: 2.2210

Critical Depth: 1.3451 ft

Critical Velocity: 4.6536 ft/s

Critical Slope: 0.0219 ft/ft

Critical Top Width: 8.07 ft

Calculated Max Shear Stress: 7.3199 lb/ft^2

Calculated Avg Shear Stress: 3.4721 lb/ft^2

ATTACHMENT C

Permissible Shear Stress

Job: Scepter West Phase Major Mod
Project No.: 60398526
Description: Letdown Permissible Shear Stress
Computed by: N. Popkowski Date: 3/15/17
Checked by: M. May Date: 3/15/17

Reference

① Determine Permissible Shear Stress of TDOT Machined Riprap (Class A-1)

$$\tau_p = F^*(\gamma_s - \gamma) D_{50} \leftarrow \text{HEC-15 Chapter 6}$$

where, τ_p = permissible shear stress (lb/ft^2)
F^* = shield's parameter (dimensionless) = 0.047
γ_s = specific weight of stone (lb/ft^3) = 165 lb/ft^3
γ = specific weight of water (lb/ft^3) = 62.4 lb/ft^3
D_{50} = mean riprap size (ft) = 9 in = 0.75 ft

$$\tau_p = 0.047(165 \text{ lb/ft}^3 - 62.4 \text{ lb/ft}^3)\, 0.75 \text{ ft}$$

$$\underline{\tau_p = 3.62 \text{ lb/ft}^2}$$

APPENDIX F.2

Leachate Management Calculations

F.2A Alternative Liner System Demonstration

F.2B HELP Model and Leachate Generation Estimate Rev 1

F.2C Liner System Geocomposite Design Rev 1

F.2D Pipe Loading Calculations

F.2E Protective Cover Graded Filter Evaluation

F.2.F Leachate Secondary Containment

APPENDIX F.2A

Alternative Liner System Demonstration

I. Objective

The purpose of this analysis is to evaluate the hydraulic equivalency between two potential liner systems for the future expansion of the Scepter Landfill. The two potential liner systems (Liner System 1 and Liner System 2) both consist of drainage layer (geocomposite or granular) overlying a geosynthetic membrane liner (geomembrane). However, in Liner System 1, the geomembrane is underlain by a compacted clay liner (CCL), and in Liner System 2, the geomembrane is underlain by a geosynthetic clay liner (GCL). Liner System 2 would also include a 2 ft recompacted soil layer on which the GCL would be placed. However, inclusion of this additional barrier soil in the demonstration of hydraulic equivalency is not necessary as is presented in the conclusion section of the analysis.

The following sections summarize the pertinent solid waste regulations, the methodology, input parameters/assumptions, results, and conclusions of the liner equivalency analysis. For further detail on the specific calculations performed, refer to the corresponding input/output data provided in the attachments.

II. Class I and Class II Liner/Geologic Buffer System Requirements

This section identifies the requirements specific to liners and geologic buffers as found in Rule 0400-11-01-.04.

(4) Leachate Migration Control Standards

(a) Class I Disposal Facilities

1. Such facilities must have a liner designed to function for the estimated life of the site and the postclosure care period. It shall be designed, constructed, and installed to ensure that the concentration values listed in Appendix III of this rule will not be exceeded in the uppermost aquifer at the relevant point of compliance. The liner must be:

(i) A composite liner consisting of two components; the upper component must consist of a minimum 30-mil flexible membrane liner (FML), and the lower component must consist of at least a two-foot layer of compacted soil with a hydraulic conductivity of no more than 1.0 x l0-7 cm/sec. FML components consisting of high density polyethylene (HDPE) shall be at least 60-mil thick. The FML component must be installed in direct and uniform contact with the compacted soil component;

2. Underlying the liners shall be a geologic buffer which shall have:

(i) A maximum hydraulic conductivity of 1.0 x 10-5 cm/s and measures at least ten (10) feet from the bottom of the liner to the seasonal high water table of the uppermost unconfined aquifer or the top of the formation of a confined aquifer or

(ii) Have a maximum hydraulic conductivity of 1.0 x 10-6 cm/s and measures not less than five(5) feet from the bottom of the liner to the seasonal high water table of the uppermost unconfined aquifer or the top of the formation of a confined aquifer or

4. Alternate liner designs may be used provided that:

(i) It is demonstrated to the satisfaction of the Commissioner that the liner design provides equivalent or superior performance to the minimum performance standard for a Class I facility as described in this subparagraph, and

(ii) When approving a design the Commissioner, shall consider at least the following factors:

(I) The hydrogeologic characteristics of the facility and surrounding land;

(II) The climatic factors of the area; and

(III) The volume and physical and chemical characteristics of the leachate.

III. Methodology

For Subtitle D landfills the regulatory allowable head acting on the liner system is 30 cm. In predicting leakage flow rate, this is a conservative case, compared to performing a HELP model analysis.

For estimation of leakage rate, the composite liner system consists of a geomembrane and a low-permeability soil component (CCL or GCL). With a composite barrier, the flow through a penetration in the geomembrane is reduced by the underlying clay component. Giroud (1997) developed semi-empirical equations to estimate the leakage rate through a composite liner system. Note that due to the semi-empirical nature of the equation, calculations must be performed with SI units as presented below. The leakage rate through a composite liner system with a circular defect is estimated as follows:

$$Q = C_{qo}\left[1 + 0.1\left(\frac{h}{t_s}\right)^{0.95}\right] a^{0.1} h^{0.9} k_s^{0.74}, \quad \text{where,}$$

Q	=	Leakage rate through composite system (m^3/sec)
C_{qo}	=	Contact quality factor (dimensionless)
H	=	Head of liquid above the top of the geomembrane (m)
t_s	=	Thickness of low-permeability soil component (m)
A	=	Area of defect in geomembrane (m^2)

k_s = Hydraulic conductivity of low-permeability soil component (m/s)

The leakage rate was calculated through the two composite liner systems under the following two scenarios:

- Scenario 1: A typical scenario, with the depth of water above the geomembrane equal to the thickness of an overlying geocomposite drainage material.
- Scenario 2: A scenario assuming the depth of water above the geomembrane is equal to the thickness of a 1 foot granular drainage layer, which again is a conservative case.

The leakage rate of Liner System 1, utilizing a CCL as the low-permeability soil component, will be compared to the leakage rate of Liner System 2, utilizing a GCL as the low-permeability soil component. Comparison of leakage rates will demonstrate whether Liner System 2 can be considered at least hydraulically equivalent to Liner System 1.

IV. Input Parameters/Assumptions

The input parameters provided in the table below were used for the analysis. A brief discussion of the input parameters is provided.

PARAMETER	VALUE	
C_{qo}	0.21	
a	0.0002 m^2	
h	0.0305 m (0.1 ft)	Scenario 1
	0.305 m (1 ft)	Scenario 2
t_s	0.610 m (2 ft)	Liner System 1 - CCL
	0.005 m (0.016 ft)	Liner System 2 - GCL
k_s	1 x 10^{-9} m/s	Liner System 1 - CCL
	5 x 10^{-11}m/s	Liner System 2 - GCL

For Liner System 1 and Liner System 2, the height of the geocomposite (used as the head of liquid in Scenario 1), the contact conditions, and the area of the defect (2 cm^2) were assumed equal. Note that the only difference between Scenario 1 and 2 is the head of liquid above the geomembrane liner. The geocomposite drainage layer is assumed to have a thickness of 0.0305m (0.1ft), typical for a geocomposite. The thickness of the drainage layer is only used to represent the head of the liquid in Scenario 1. In the Giroud equation presented in Section III, the contact quality can be described as “good” or “poor.” Good contact is described as a geomembrane installed with few wrinkles above an adequately compacted, smooth low-permeability soil layer. Additional research has shown that it may be possible to achieve an “excellent” contact (corresponding to a C_{qo}=0.05) for a GCL in contact with a geomembrane, partially due to the inherent flat installation method, and the potential for slurry to exude from a hydrated GCL (Harper et al., 1993). For the purposes of this evaluation, "good" contact was assumed for both liner systems to be conservative, corresponding to a C_{qo}=0.21.

The thickness of the low-permeability soil, t_s, and permeability, k_s, are dependent on the liner system. For Liner System 1, the low-permeability soil is comprised of 2 ft of CCL, having a maximum permeability of 1 x 10-9 m/s (=1 x 10^{-7} cm/s). The low-permeability soil in Liner System 2 is comprised of a GCL. The specific GCL has not been selected for the proposed expansion. As a result, minimum values of thickness and permeability found in literature (Girioud, 1997) were used in the analysis.

V. Results

As stated previously, the leakage rate of Liner System 1, utilizing a CCL as the low-permeability soil component, will be compared to the leakage rate of Liner System 2, utilizing a GCL as the low-permeability soil component.

The table below summarizes the results of the analysis. The calculations are included in Appendix A.

	Q Liner System 1 (m^3/s)	Q Liner System 2 (m^3/s)	$Q_{(System\ 1)}/Q_{(System\ 2)}$
Scenario 1	8.52×10^{-10}	1.44×10^{-10}	5.9
Scenario 2	7.08×10^{-9}	4.37×10^{-9}	1.6

From the above table, under both Scenarios, the liner system using GCL as the low-permeability soil component, Liner System 2, yields a lower leakage rate.

VI. Conclusions

The results of the analysis indicate that under conditions represented by Scenario 1, the leakage rate for Liner System 1 (utilizing 2 ft of low permeability CCL) is more than 5.9 times the leakage rate through Liner System 2 (utilizing a GCL). When the head above the liner is increased to 1 foot as represented in Scenario 2, the leakage rate through the Liner System 1 is still approximately 1.6 times the leakage rate expected through Liner System 2. Based on these results, the proposed Liner System 2, utilizing GCL is considered to be at least hydraulically equivalent Liner System 1, with respect to preventing infiltration. Furthermore, Liner System 2 would also consist of a 2 foot thick underlying recompacted soil layer. As shown from the analysis, this recompacted soil layer provides additional protection, but does not factor into the equivalency demonstration.

The Project Specifications will incorporate GCL liner meeting at least the minimum criterion stated in the analyses (t_s = 5 mm, k_s = 5 x 10^{-11} m/s).

VII. References/Software

The following references and software were utilized for the calculations:

Giroud, J.P. "Equations for Calculating the Rate of Liquid Migration Through Composite Liners Due to Geomembrane Defects", *Geosynthetics International*, Vol. 4, Nos. 3-4, pp.335-348, 1997.

Giroud, J. P., and Bonaparte, R. (1989). "Leakage through liners constructed with geomembrane liners--parts I and II and technical note," Geotextiles and Geomembranes 8(1), 27-67, 8(2), 71-111, 8(4), 337-340.

Harpur, W.A., R.F. Wilson-Fahmy, and R.M. Koerner. (1993). “Evaluation of the contact between geosynthetic clay liners and geomembranes in terms of transmissivity. *Proceedings of the 7th Annual GRI Seminar*. Drexel University, Philadelphia, PA, United States. 138.149.

Software – Microsoft Excel (2010)

Leakage Through a Hole in the Geomembrane - Equivalency Calculation (Giroud)

$$Q = C_{qo}\left(1 + 0.1\left(\frac{h}{t_s}\right)^{0.95}\right)a^{0.1}\ h^{0.9}\ k_s^{0.74}$$

Where:

Q = rate of liquid migration through the composite system (membrane and soil component)
C_{qo} = contact quality factor, a constant related to the level of contact between the geomembrane and underlying layer
h = head of water on top of the geomembrane (m)
t_s = thickness of the low permeability soil component of the liner (m)
a = area of defect in geomembrane (m^2)
k_s= hydraulic conductivity of the underlying low permeability soil (m/s)

Scenario 1
0.1 feet of head on the geomembrane liner
(geocomposite drainage material above liner system is saturated)

Geomembrane with two foot CCL

C_{qo} = 0.21 (assumes "good" installation - typical for clay/geomembrane)
h = 0.0305 m (0.1 feet)
t_s = 0.610 m (2 feet)
a = 0.0002 m^2 (standard industry assumption - 2 holes per acre at 0.0001 m^2 per hole)
k_s = 1.00E-09 m/s (1 x 10 $^{-7}$ cm/s)

Q_{CCL} = 8.52E-10 m^3/s 1.94E-02 gal/day

Geomembrane with GCL

C_{qo} = 0.21 (assumes "good" installation)
h = 0.0305 m (0.1 foot)
t_s = 0.005 m (5 mm)
a = 0.0002 m^2 (standard industry assumption - 2 holes per acre at 0.0001 m^2 per hole)
k_s = 5.00E-11 m/s (5.0x 10 $^{-9}$ cm/s)

Q_{GCL} = 1.44E-10 m^3/s 3.28E-03 gal/day

Q_{CCL}/Q_{GCL} = 5.93 **- CCL leaks 5.93 x more than the GCL**

Scenario 2
1 foot of head on the geomembrane liner
(1 foot of material above geocomposite drainage material is saturated)

Geomembrane with two foot CCL

C_{qo} = 0.21 (assumes "good" installation - typical for clay/geomembrane)
h = 0.305 m (1 foot)
t_s = 0.610 m (2 feet)
a = 0.0002 m^2 (standard industry assumption - 2 holes per acre at 0.0001 m^2 per hole)
k_s = 1.00E-09 m/s (1 x 10 $^{-7}$ cm/s)

Q_{CCL} = 7.08E-09 m^3/s 1.62E-01 gal/day

Geomembrane with GCL

C_{qo} = 0.21 (assumes "good" installation)
h = 0.305 m (1 foot)
t_s = 0.005 m (5 mm)
a = 0.0002 m^2 (standard industry assumption - 2 holes per acre at 0.0001 m^2 per hole)
k_s = 5.00E-11 m/s (5.0 x 10 $^{-9}$ cm/s)

Q_{GCL} = 4.37E-09 m^3/s 9.98E-02 gal/day

Q_{CCL}/Q_{GCL} = 1.62 **- CCL leaks 1.62 x more than GCL**

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615-771-2459 fax

September 28, 2017

Mr. Jason Repsher, Nashville Field Office Manager
Tennessee Department of Environment & Conservation Nashville EFO
711 R.S. Gass Blvd.
Nashville, TN 37243

RE: GCL Compatibility Testing
Part II Permit Application
Scepter Disposal Facility – East Phase Site
Humphreys County, Tennessee

Dear Mr. Repsher:

The attached GCL compatibility test report has been provided by GSE Environmental. One of their polymer enhanced GCL products has demonstrated compatibility with Scepter site specific leachate in accordance with ASTM D-6766. Compatible means the GCL hydraulic conductivity is equal to or less than the maximum permissible permeability while the three ASTM D-6766 termination criteria were met.

As the attached report shows, this particular GCL has met the ASTM D-6766 termination criteria with a hydraulic conductivity of 3.5×10^{-10} cm/s. AECOM on behalf of Scepter Inc., would like TDEC's approval for this particular GCL formulation and any other GCL product formulations that are demonstrated through similar testing to achieve a maximum permeability of 9.59×10^{-9} cm/s (see attachment 1 for maximum permissible permeability that is equivalent to a CCL).

Sincerely,

AECOM

Michael May

Michael May, P.E.
Project Manager

Cc: Mr. Brian Griffin, Scepter, Inc.

19103 Gundle Rd
Houston, TX 77073

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GCL Compatibility Testing
Polymer-Enhanced BentoLiner GCL Exposed to A Site-Specific Leachate

East Phase Landfill
Scepter, Inc.
Waverly, Tennessee

October 31, 2017
Rev. 1

This report summarizes the test results for leachate chemical compatibility testing of GSE's Polymer-Enhanced (PE) Geosynthetic Clay Liner (GCL) product family, represented in this case by BentoLiner AR GCL, when exposed to a site-specific leachate for the East Phase Landfill at Scepter, Inc. (Waverly, TN).

GCL is an active barrier system which relies on the swelling capacity of the sodium bentonite to decrease flow through the liner. Sodium bentonite can be affected by various ions which can sometimes be present in industrial leachate solutions. These ions can negatively impact GCL performance by increasing the product's hydraulic conductivity, and consequently alter the hydraulic equivalence between the GCL and compacted clay liner (CCL).

The PE BentoLiner GCL product family incorporates polymer enhanced sodium bentonite to improve GCL hydraulic performance. Leachate generated from aluminum processing waste has its own distinct chemical characterization. Because of this, GSE typically recommends that the GCL be tested per ASTM D 6766, Scenario 2 to confirm chemical compatibility with site specific leachate. In this case, the test was performed with site specific leachate provided to GSE by Scepter, Inc. (Waverly, TN).

The leachate was sent to an independent lab (Virginia Geoenvironmental I Laboratory, University of Virginia, Charlottesville, VA) for a general chemical analysis of the cations and anions. A listing of the key constituents of concern is listed in Table 1.

Table 1: Leachate Chemical Summary (mg/l)

Site Leachate	Calcium	Magnesium	Potassium	Sodium	Chlorides	Sulfates	pH
Scepter East	0.1	0.1	54,410	79,320	152,810	3,430	9.4

The leachate was tested at an independent laboratory (Wisconsin Geotechnical Laboratory, University of Wisconsin, Madison, WI) for chemical compatibility testing by performing hydraulic conductivity tests. All hydraulic conductivity testing was performed in general accordance with Scenario 2 in ASTM D6766, *Standard Test Method for Evaluation of Hydraulic Properties of Geosynthetic Clay Liners Permeated with Potentially Incompatible Liquids.*

Wisconsin Geotechnical Laboratory used a flexible wall permeameter to perform the hydraulic conductivity testing. The test used typical ASTM D6766 setup parameters, and the sample was hydrated for 48 hours with the site-specific leachate prior to permeation without a hydraulic gradient and with the effective stress applied. Backpressure was not used to simulate the field condition and to prevent chemical alterations of the permeant liquid. After the hydration period, the leachate flowed through the GCL. This test was performed for 181 days. The test reached steady state flow conditions at about 3.6 pore volumes and the specimen reached chemical equilibrium. ASTM D6766 termination criteria were satisfied. Table 2 summarizes the test results.

GSE ENVIRONMENTAL™

Table 2: Summary of Hydraulic Conductivity Performance against Site Specific Leachate

Site Leachate	Testing Period (Days)	BentoLiner AR GCL (cm/sec)	Total Inflow Volume (cc)
Scepter East	181	3.5×10^{-10}	206

In conclusion, the leachate chemical compatibility test was performed for 181 days demonstrating a hydraulic conductivity of 3.5×10^{-10} cm/sec upon completion. As required by project specifications, BentoLiner AR GCL decreases in hydraulic conductivity over time and is well below the required maximum hydraulic conductivity performance. These test results demonstrate that BentoLiner AR GCL, as manufactured by GSE Environmental, LLC, is chemically compatible with this site-specific leachate. The enclosed data sheet has additional information on the test outcome.

Respectfully,

Steven Mayes, P.E.
Technical Director & Southeast Region Sales Manager

Wisconsin Geotechnical Laboratory

GCL Hydraulic Compatibility Test with Leachate per ASTM D6766

Sample ID:	AL-AQE-AS-S		**Test Date :**	1/30/17	
GCL:	Mock with AL-AQE-AS-S		Permeant Liquid:	Scepter East	
Cell Pressure:	4.0	psi	Diameter of Sample, D:	15.2	cm
Backpressure:	0.0	psi	Length of Sample, L:	0.5	cm
Head Setup Constant:	126.0	cm	Area of Sample, A:	182.4	cm^2
Effective Stress:	4.0	psi	Burette Area, a:	1.0	cm^2
Max. Hydraulic Gradient:	252	-	Min. Hydraulic Gradient :	152.0	

Dry Wt Bentonite:	73	g
Sample Volume, V:	91.2	mL
Unit Weight, γ:	1.89	Mg/m^3
Dry Unit Weight, γ_d:	0.80	Mg/m^3
Void Ratio, e:	2.31	-
Porosity, n:	0.70	-

Post Test Conditions:

GCL Thickness :	5.0	mm
Wet Sample Weight:	172.2	g
Final Water Content:	80.3	%
Ash Content After Test:	91.1	%
LOI After Test:	8.9	%

$$K_s = \frac{a * L}{A * \Delta t} Ln\left\{\frac{H_1}{H_2}\right\}$$

Date, Time	Burette Reading		Δt (s)	H (cm)	El. Time (hour)	K (cm/s)	Q_{out} / Q_{in}	Incremental Volume Q_{in} mL	Incremental Volume Q_{out} mL	Influent EC mS/cm	Influent pH -	Effluent EC mS/cm	Effluent pH -	EC Ratio -	Total Q_{out} mL	Cum. Inflow	PVF Inflow
2/1/17 9:36	0.0		0	126.0	0.0					291.1	9.2				0	0	
2/2/17 10:03	8.5	7	88020	117.5	24.4	2.17E-09	0.82	8.50	7.00						7	8.50	0.1335
2/3/17 9:28	11.5	7	84300	114.5	47.9	8.41E-10	0.00	3.00	0.00						7	11.50	0.1806
2/5/17 9:55	15.9	7	174420	110.1	96.3	6.16E-10	0.00	4.40	0.00						7	15.90	0.2498
2/8/17 9:59	20.9		259440	105.1	168.4	4.91E-10	0.00	5.00	0.00						7	20.90	0.3283
2/11/17 14:36	25.6		275820	100.4	245.0	4.55E-10	0.00	4.70	0.00						7	25.60	0.4021
2/13/17 10:23	28.1		157620	97.9	288.8	4.38E-10	0.00	2.50	0.00						7	28.10	0.4414
2/18/17 14:52	34.3		448140	91.7	413.3	4.00E-10	0.00	6.20	0.00						7	34.30	0.5388
2/25/17 16:16	39.5	12	609840	86.5	582.7	2.62E-10	2.31	5.20	12.00						19	39.50	0.6205
2/25/17 16:16	0.0	12	0	126.0	582.7		0.00	0.00	0.00						19	39.50	0.6205
3/1/17 11:11	9.5	15	327300	116.5	673.6	6.56E-10	0.32	9.50	3.00						22	49.00	0.7697
3/4/17 15:49	14.5	19	275880	111.5	750.2	4.36E-10	0.80	5.00	4.00						26	54.00	0.8483
3/13/17 10:09	25.4	30	757200	100.6	960.6	3.72E-10	1.01	10.90	11.00						37	64.90	1.0195
3/24/17 10:26	35.6	40	951420	90.4	1224.8	3.08E-10	0.98	10.20	10.00						47	75.10	1.1797
3/31/17 10:34	41.5	46	605280	84.5	1393.0	3.06E-10	1.02	5.90	6.00	297.1	9.1	290.4	6.6	0.98	53	81.00	1.2724
3/31/17 10:34	0.0	0	0	126.0	1393.0										53	81.00	1.2724
4/4/17 11:01	9.6	7	347220	116.4	1489.4	6.26E-10	0.73	9.60	7.00						60	90.60	1.4232
4/11/17 11:11	19.0	17	605400	107.0	1657.6	3.81E-10	1.06	9.40	10.00						70	100.00	1.5708
4/22/17 9:13	33.2	33	943320	92.8	1919.6	4.14E-10	1.13	14.20	16.00						86	114.20	1.7939
4/28/17 9:58	37.0	37	521100	89.0	2064.4	2.20E-10	1.05	3.80	4.00	297.1	9.1	290.8	8.1	0.98	90	118.00	1.8536
4/28/17 9:58	0.0	0	0	126.0	2064.4										90	118.00	1.8536
5/5/17 13:08	13.7	11	616200	112.3	2235.5	5.12E-10	0.80	13.70	11.00						101	131.70	2.0688
5/12/17 11:49	22.5	21	600060	103.5	2402.2	3.73E-10	1.14	8.80	10.00						111	140.50	2.2070
5/22/17 10:01	33.0	32	857520	93.0	2640.4	3.42E-10	1.05	10.50	11.00						122	151.00	2.3720
5/30/17 14:36	40.2	40	707700	85.8	2837.0	3.12E-10	1.11	7.20	8.00	297.1	9.1	294.9	9.1	0.99	130	158.20	2.4851
5/30/17 14:36	0.0	0	0	126.0	2837.0										130	158.20	2.4851
6/12/17 9:08	20.5	20	1103520	105.5	3143.5	4.41E-10	0.98	20.50	20.00						150	178.70	2.8071
6/19/17 10:09	28.0	28	608460	98.0	3312.6	3.32E-10	1.07	7.50	8.00						158	186.20	2.9249

6/30/17 11:22	38.0	38	954780	88.0	3577.8	3.09E-10	1.00	10.00	10.00						168	196.20	3.0820
7/5/17 9:46	41.7	42	426240	84.3	3696.2	2.76E-10	1.08	3.70	4.00						172	199.90	3.1401
7/7/17 10:13	43.2	43	174420	82.8	3744.6	2.82E-10	0.67	1.50	1.00						173	201.40	3.1637
7/10/17 9:31	45.3	45	256680	80.7	3815.9	2.74E-10	0.95	2.10	2.00	297.1	9.1	295.0	9.1	0.99	175	203.50	3.1967
7/10/17 9:31	0.0	0	0	126.0	3815.9										175	203.50	3.1967
7/17/17 9:12	13.5	13	603660	112.5	3983.6	5.15E-10	0.96	13.50	13.00						188	217.00	3.4087
7/26/17 10:04	24.5	25	780720	101.5	4200.5	3.61E-10	1.09	11.00	12.00						200	228.00	3.5815
7/27/17 14:36	25.8	26.5	102720	100.2	4229.0	3.44E-10	1.15	1.30	1.50	297.1	9.1	296.3	9.2	1.00	202	229.30	3.6019
7/31/17 14:04	29.9	31	343680	96.1	4324.5	3.33E-10	1.10	4.10	4.50	297.1	9.1	294.0	9.3	0.99	206	233.40	3.6663

K (cm/s) = 3.5E-10

Leakage Through a Hole in the Geomembrane - Equivalency Calculation (Giroud)

$$Q = C_{qo}\left(1 + 0.1\left(\frac{h}{t_s}\right)^{0.95}\right) a^{0.1}\ h^{0.9}\ k_s^{0.74}$$

Where:

Q = rate of liquid migration through the composite system (membrane and soil component)
C_{qo} = contact quality factor, a constant related to the level of contact between the geomembrane and underlying layer
h = head of water on top of the geomembrane (m)
t_s = thickness of the low permeability soil component of the liner (m)
a = area of defect in geomembrane (m^2)
k_s= hydraulic conductivity of the underlying low permeability soil (m/s)

Scenario 1
0.1 feet of head on the geomembrane liner
(geocomposite drainage material above liner system is saturated)

Geomembrane with two foot CCL

C_{qo} = 0.21 (assumes "good" installation - typical for clay/geomembrane)
h = 0.0305 m (0.1 feet)
t_s = 0.610 m (2 feet)
a = 0.0002 m^2 (standard industry assumption - 2 holes per acre at 0.0001 m^2 per hole)
k_s = 1.00E-09 m/s (1 x 10^{-7} cm/s)

Q_{CCL} = 8.52E-10 m^3/s 1.94E-02 gal/day

Geomembrane with GCL

C_{qo} = 0.21 (assumes "good" installation)
h = 0.0305 m (0.1 foot)
t_s = 0.005 m (5 mm)
a = 0.0002 m^2 (standard industry assumption - 2 holes per acre at 0.0001 m^2 per hole)
k_s = 5.54E-10 m/s (5.54x 10^{-8} cm/s)

Q_{GCL} = 8.52E-10 m^3/s 1.95E-02 gal/day

Q_{CCL}/Q_{GCL} = 1.00 **- CCL leakage = GCL leakage**

Scenario 2
1 foot of head on the geomembrane liner
(1 foot of material above geocomposite drainage material is saturated)

Geomembrane with two foot CCL

C_{qo} = 0.21 (assumes "good" installation - typical for clay/geomembrane)
h = 0.305 m (1 foot)
t_s = 0.610 m (2 feet)
a = 0.0002 m^2 (standard industry assumption - 2 holes per acre at 0.0001 m^2 per hole)
k_s = 1.00E-09 m/s (1 x 10^{-7} cm/s)

Q_{CCL} = 7.08E-09 m^3/s 1.62E-01 gal/day

Geomembrane with GCL

C_{qo} = 0.21 (assumes "good" installation)
h = 0.305 m (1 foot)
t_s = 0.005 m (5 mm)
a = 0.0002 m^2 (standard industry assumption - 2 holes per acre at 0.0001 m^2 per hole)
k_s = 9.59E-11 m/s (9.59 x 10^{-9} cm/s)

Q_{GCL} = 7.08E-09 m^3/s 1.62E-01 gal/day

Q_{CCL}/Q_{GCL} = 1.00 **- CCL leakage = GCL leakage**

APPENDIX F.2B

HELP Model and Leachate Generation Estimate
Rev 1

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**
 **             HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE
**
 **               HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)
**
 **                 DEVELOPED BY ENVIRONMENTAL LABORATORY
**
 **                   USAE WATERWAYS EXPERIMENT STATION
**
 **            FOR USEPA RISK REDUCTION ENGINEERING LABORATORY
**
 **
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 PRECIPITATION DATA FILE:      z:\nick\SPTR_PRE.D4
 TEMPERATURE DATA FILE:        z:\nick\SPTR_TEM.D7
 SOLAR RADIATION DATA FILE:    z:\nick\SPTR_SOL.D13
 EVAPOTRANSPIRATION DATA:      z:\nick\SPTR-EVP.D11
 SOIL AND DESIGN DATA FILE:    z:\nick\SPTR_OP2.D10
 OUTPUT DATA FILE:             z:\nick\SPTR_OP2.OUT


 TIME:   6:14     DATE:  10/18/2017



***************************************************************************
*****

      TITLE:  Scepter Waverly East Phase Landfill: Open Condition
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     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
            COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                               LAYER  1
                               --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  24
      THICKNESS                    =      6.00   INCHES
      POROSITY                     =      0.3650 VOL/VOL
      FIELD CAPACITY               =      0.3050 VOL/VOL
      WILTING POINT                =      0.2020 VOL/VOL
      INITIAL SOIL WATER CONTENT   =      0.3568 VOL/VOL
      EFFECTIVE SAT. HYD. COND.    =  0.270000010000E-05 CM/SEC


                               LAYER  2
                               --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER   0
      THICKNESS                    =    120.00   INCHES
      POROSITY                     =      0.3750 VOL/VOL
      FIELD CAPACITY               =      0.0550 VOL/VOL
      WILTING POINT                =      0.0200 VOL/VOL
      INITIAL SOIL WATER CONTENT   =      0.1280 VOL/VOL
      EFFECTIVE SAT. HYD. COND.    =  0.100000005000E-02 CM/SEC


                               LAYER  3
                               --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                      MATERIAL TEXTURE NUMBER  10
      THICKNESS                    =     12.00   INCHES
      POROSITY                     =      0.3980 VOL/VOL
      FIELD CAPACITY               =      0.2440 VOL/VOL
      WILTING POINT                =      0.1360 VOL/VOL
      INITIAL SOIL WATER CONTENT   =      0.2440 VOL/VOL
      EFFECTIVE SAT. HYD. COND.    =  0.119999997000E-03 CM/SEC
```

LAYER 4

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 1

THICKNESS	=	12.00 INCHES
POROSITY	=	0.4170 VOL/VOL
FIELD CAPACITY	=	0.0450 VOL/VOL
WILTING POINT	=	0.0180 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0450 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.999999978000E-02 CM/SEC

LAYER 5

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20 INCHES
POROSITY	=	0.8500 VOL/VOL
FIELD CAPACITY	=	0.0100 VOL/VOL
WILTING POINT	=	0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000 CM/SEC
SLOPE	=	3.00 PERCENT
DRAINAGE LENGTH	=	200.0 FEET

LAYER 6

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	3.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD

LAYER 7

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #24 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 5.% AND A SLOPE LENGTH OF 200. FEET.

SCS RUNOFF CURVE NUMBER	=	96.90	
FRACTION OF AREA ALLOWING RUNOFF	=	0.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.141	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	2.190	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.212	INCHES
INITIAL SNOW WATER	=	6.567	INCHES
INITIAL WATER IN LAYER MATERIALS	=	21.157	INCHES
TOTAL INITIAL WATER	=	27.724	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM NASHVILLE TENNESSEE

STATION LATITUDE	=	36.12	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	86	
END OF GROWING SEASON (JULIAN DATE)	=	308	
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.00	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	68.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	69.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	75.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	71.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR NASHVILLE TENNESSEE

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.49	4.03	5.58	4.47	4.56	3.70
3.82	3.40	3.71	2.58	3.52	4.63

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR NASHVILLE TENNESSEE

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
37.10	40.40	49.00	59.60	68.10	75.80
79.40	78.40	72.30	60.20	48.60	40.90

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR NASHVILLE TENNESSEE AND STATION LATITUDE = 36.12 DEGREES

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 100

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	4.31	4.65	5.61	4.43	4.53	3.61
	3.89	3.49	3.94	2.43	3.57	4.50

```
    STD. DEVIATIONS          1.84     2.17     2.23     1.81     2.12
1.63
                             2.05     2.00     2.35     1.59     1.68
2.25

  RUNOFF
  ------
    TOTALS                   0.000    0.000    0.000    0.000    0.000
0.000
                             0.000    0.000    0.000    0.000    0.000
0.000

    STD. DEVIATIONS          0.000    0.000    0.000    0.000    0.000
0.000
                             0.000    0.000    0.000    0.000    0.000
0.000

  EVAPOTRANSPIRATION
  ------------------
    TOTALS                   0.839    1.089    2.175    2.794    3.598
3.974
                             3.764    3.384    2.513    1.836    1.127
0.788

    STD. DEVIATIONS          0.430    0.605    1.482    2.404    3.191
3.553
                             3.456    3.101    2.286    1.653    0.862
0.467

  LATERAL DRAINAGE COLLECTED FROM LAYER  5
  ----------------------------------------
    TOTALS                   1.3065   1.4347   1.6931   1.9131   2.0609
1.9430
                             1.9237   1.8066   1.6583   1.6459   1.5802
1.4852

    STD. DEVIATIONS          0.8613   0.6914   0.7328   0.7876   0.8528
0.8768
                             0.9793   0.9932   0.9989   1.0681   1.0466
1.0935

  PERCOLATION/LEAKAGE THROUGH LAYER  7
  ------------------------------------
    TOTALS                   0.0000   0.0000   0.0000   0.0000   0.0000
0.0000
                             0.0000   0.0000   0.0000   0.0000   0.0000
0.0000

    STD. DEVIATIONS          0.0000   0.0000   0.0000   0.0000   0.0000
0.0000
                             0.0000   0.0000   0.0000   0.0000   0.0000
0.0000
```

```
 -------------------------------------------------------------------------
-------
                AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)
 -------------------------------------------------------------------------
-------


   DAILY AVERAGE HEAD ON TOP OF LAYER  6
   -------------------------------------
     AVERAGES                   0.0063   0.0075   0.0081   0.0095   0.0099
0.0096
                                0.0092   0.0087   0.0082   0.0079   0.0078
0.0071

     STD. DEVIATIONS            0.0041   0.0036   0.0035   0.0039   0.0041
0.0044
                                0.0047   0.0048   0.0050   0.0051   0.0052
0.0053

*************************************************************************
******



*************************************************************************
******

      AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS    1 THROUGH
100
 -------------------------------------------------------------------------
-------
                                           INCHES           CU. FEET
PERCENT
                                     -------------------   -------------   ---
------
  PRECIPITATION                        48.97    (    7.932)     177771.3
100.00

  RUNOFF                                0.000   (  0.0000)          0.00
0.000

  EVAPOTRANSPIRATION                   27.881   ( 21.6037)     101209.39
56.932

  LATERAL DRAINAGE COLLECTED           20.45108 (  8.33635)     74237.414
41.76006
    FROM LAYER  5

  PERCOLATION/LEAKAGE THROUGH           0.00000 (  0.00000)         0.006
0.00000
    LAYER  7
```

AVERAGE HEAD ON TOP OF LAYER 6   0.008 (0.003)

CHANGE IN WATER STORAGE   0.575   (29.4492)   2086.60
1.174

****

```
**************************************************************************
*****

               PEAK DAILY VALUES FOR YEARS    1 THROUGH  100
    ----------------------------------------------------------------------
---
                                                  (INCHES)      (CU. FT.)
                                                 ----------   ------------
-
       PRECIPITATION                               5.65         20509.500

       RUNOFF                                      0.000           0.0000

       DRAINAGE COLLECTED FROM LAYER  5            0.16210
588.41638

       PERCOLATION/LEAKAGE THROUGH LAYER  7        0.000000
0.00004

       AVERAGE HEAD ON TOP OF LAYER  6             0.025

       MAXIMUM HEAD ON TOP OF LAYER  6             0.037

       LOCATION OF MAXIMUM HEAD IN LAYER  5
             (DISTANCE FROM DRAIN)                 4.3 FEET

       SNOW WATER                                365.97       1328476.3700


       MAXIMUM VEG. SOIL WATER (VOL/VOL)                   0.3650

       MINIMUM VEG. SOIL WATER (VOL/VOL)                   0.2020

        ***  Maximum heads are computed using McEnroe's equations.  ***

             Reference:  Maximum Saturated Depth over Landfill Liner
                         by Bruce M. McEnroe, University of Kansas
                         ASCE Journal of Environmental Engineering
                         Vol. 119, No. 2, March 1993, pp. 262-270.


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FINAL WATER STORAGE AT END OF YEAR 100

LAYER	(INCHES)	(VOL/VOL)
1	2.1483	0.3581
2	20.8557	0.1738
3	3.5185	0.2932
4	1.4442	0.1204
5	0.0020	0.0100
6	0.0000	0.0000
7	0.1800	0.7500
SNOW WATER	57.057	

****

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*****
 **
**
 **
**
 **             HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE
**
 **              HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)
**
 **               DEVELOPED BY ENVIRONMENTAL LABORATORY
**
 **                 USAE WATERWAYS EXPERIMENT STATION
**
 **            FOR USEPA RISK REDUCTION ENGINEERING LABORATORY
**
 **
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 PRECIPITATION DATA FILE:     z:\nick\SPTR_PRE.D4
 TEMPERATURE DATA FILE:       z:\nick\SPTR_TEM.D7
 SOLAR RADIATION DATA FILE:   z:\nick\SPTR_SOL.D13
 EVAPOTRANSPIRATION DATA:     z:\nick\SPTR-EVP.D11
 SOIL AND DESIGN DATA FILE:   z:\nick\SPTR_INT.D10
 OUTPUT DATA FILE:            z:\nick\SPTR_INT.OUT


 TIME:  19:22     DATE:  10/19/2017


***************************************************************************
*****

     TITLE:  Scepter Waverly East Phase Landfill: Intermediate Condition
```

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****************************************************************************
*****


    NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
           COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                              LAYER  1
                              --------

                   TYPE 1 - VERTICAL PERCOLATION LAYER
                       MATERIAL TEXTURE NUMBER  24
          THICKNESS                    =     12.00    INCHES
          POROSITY                     =      0.3650 VOL/VOL
          FIELD CAPACITY               =      0.3050 VOL/VOL
          WILTING POINT                =      0.2020 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.3452 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.270000010000E-05 CM/SEC


                              LAYER  2
                              --------

                   TYPE 1 - VERTICAL PERCOLATION LAYER
                       MATERIAL TEXTURE NUMBER   0
          THICKNESS                    =    960.00    INCHES
          POROSITY                     =      0.3750 VOL/VOL
          FIELD CAPACITY               =      0.0550 VOL/VOL
          WILTING POINT                =      0.0200 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.0557 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.100000005000E-02 CM/SEC


                              LAYER  3
                              --------

                   TYPE 1 - VERTICAL PERCOLATION LAYER
                       MATERIAL TEXTURE NUMBER  10
          THICKNESS                    =     12.00    INCHES
          POROSITY                     =      0.3980 VOL/VOL
          FIELD CAPACITY               =      0.2440 VOL/VOL
          WILTING POINT                =      0.1360 VOL/VOL
          INITIAL SOIL WATER CONTENT   =      0.2440 VOL/VOL
          EFFECTIVE SAT. HYD. COND.    =  0.119999997000E-03 CM/SEC
```

LAYER 4

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 1

THICKNESS	=	12.00	INCHES
POROSITY	=	0.4170	VOL/VOL
FIELD CAPACITY	=	0.0450	VOL/VOL
WILTING POINT	=	0.0180	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0450	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.999999978000E-02	CM/SEC

LAYER 5

TYPE 2 - LATERAL DRAINAGE LAYER
MATERIAL TEXTURE NUMBER 20

THICKNESS	=	0.20	INCHES
POROSITY	=	0.8500	VOL/VOL
FIELD CAPACITY	=	0.0100	VOL/VOL
WILTING POINT	=	0.0050	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0100	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	10.0000000000	CM/SEC
SLOPE	=	3.00	PERCENT
DRAINAGE LENGTH	=	200.0	FEET

LAYER 6

TYPE 4 - FLEXIBLE MEMBRANE LINER
MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06	INCHES
POROSITY	=	0.0000	VOL/VOL
FIELD CAPACITY	=	0.0000	VOL/VOL
WILTING POINT	=	0.0000	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12	CM/SEC
FML PINHOLE DENSITY	=	1.00	HOLES/ACRE
FML INSTALLATION DEFECTS	=	3.00	HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD	

LAYER 7

TYPE 3 - BARRIER SOIL LINER
MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.24 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #24 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 5.% AND A SLOPE LENGTH OF 200. FEET.

SCS RUNOFF CURVE NUMBER	=	96.90	
FRACTION OF AREA ALLOWING RUNOFF	=	75.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.066	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	2.190	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.212	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	61.278	INCHES
TOTAL INITIAL WATER	=	61.278	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
NASHVILLE TENNESSEE

STATION LATITUDE	=	36.12	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	86	
END OF GROWING SEASON (JULIAN DATE)	=	308	
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.00	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	68.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	69.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	75.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	71.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NASHVILLE TENNESSEE

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.49	4.03	5.58	4.47	4.56	3.70
3.82	3.40	3.71	2.58	3.52	4.63

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NASHVILLE TENNESSEE

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
37.10	40.40	49.00	59.60	68.10	75.80
79.40	78.40	72.30	60.20	48.60	40.90

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NASHVILLE TENNESSEE
AND STATION LATITUDE = 36.12 DEGREES

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 100

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	4.31	4.65	5.61	4.43	4.53	3.61
	3.89	3.49	3.94	2.43	3.57	4.50

STD. DEVIATIONS	1.84	2.17	2.23	1.81	2.12	1.63
	2.05	2.00	2.35	1.59	1.68	2.25

RUNOFF

TOTALS	2.869	3.222	3.843	2.565	2.776	1.988
	2.309	2.169	2.634	1.452	2.220	2.935
STD. DEVIATIONS	1.661	1.903	2.106	1.306	1.517	1.064
	1.518	1.453	1.833	1.137	1.299	1.873

EVAPOTRANSPIRATION

TOTALS	1.242	1.407	2.017	1.916	1.739	1.648
	1.524	1.326	1.273	0.991	1.180	1.149
STD. DEVIATIONS	0.303	0.396	0.577	0.657	0.762	0.719
	0.675	0.616	0.584	0.556	0.389	0.283

LATERAL DRAINAGE COLLECTED FROM LAYER 5

TOTALS	0.0093	0.0090	0.0109	0.0134	0.0153	0.0148
	0.0149	0.0137	0.0120	0.0113	0.0102	0.0100
STD. DEVIATIONS	0.0146	0.0146	0.0172	0.0195	0.0204	0.0187
	0.0183	0.0166	0.0149	0.0148	0.0144	0.0151

PERCOLATION/LEAKAGE THROUGH LAYER 7

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 6

AVERAGES	0.0000	0.0000	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0000
STD. DEVIATIONS	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001
	0.0001	0.0001	0.0001	0.0001	0.0001	0.0001

****

****

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 100

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.97 (7.932)	177771.3	100.00
RUNOFF	30.982 (5.8659)	112465.44	63.264
EVAPOTRANSPIRATION	17.412 (2.6464)	63207.37	35.555
LATERAL DRAINAGE COLLECTED FROM LAYER 5	0.14487 (0.18142)	525.862	0.29581
PERCOLATION/LEAKAGE THROUGH LAYER 7	0.00000 (0.00000)	0.003	0.00000

```
  AVERAGE HEAD ON TOP              0.000 (    0.000)
    OF LAYER  6

  CHANGE IN WATER STORAGE          0.433   (  1.1190)       1572.57
0.885

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                PEAK DAILY VALUES FOR YEARS    1 THROUGH  100
    --------------------------------------------------------------------------
---
                                                     (INCHES)      (CU. FT.)
                                                    ----------    ------------
-
       PRECIPITATION                                   5.65         20509.500

       RUNOFF                                          5.210        18910.7930

       DRAINAGE COLLECTED FROM LAYER  5                0.00323
11.72302

       PERCOLATION/LEAKAGE THROUGH LAYER  7            0.000000
0.00003

       AVERAGE HEAD ON TOP OF LAYER  6                 0.000

       MAXIMUM HEAD ON TOP OF LAYER  6                 0.001

       LOCATION OF MAXIMUM HEAD IN LAYER  5
             (DISTANCE FROM DRAIN)                     0.0 FEET

       SNOW WATER                                      7.79         28269.1699


       MAXIMUM VEG. SOIL WATER (VOL/VOL)                     0.3650

       MINIMUM VEG. SOIL WATER (VOL/VOL)                     0.2020


        ***  Maximum heads are computed using McEnroe's equations.  ***

             Reference:  Maximum Saturated Depth over Landfill Liner
                         by Bruce M. McEnroe, University of Kansas
                         ASCE Journal of Environmental Engineering
                         Vol. 119, No. 2, March 1993, pp. 262-270.


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FINAL WATER STORAGE AT END OF YEAR 100

LAYER	(INCHES)	(VOL/VOL)
1	3.3291	0.2774
2	97.1766	0.1012
3	2.9280	0.2440
4	0.9837	0.0820
5	0.0020	0.0100
6	0.0000	0.0000
7	0.1800	0.7500
SNOW WATER	0.000	

****

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APPENDIX F.2C

Liner System Geocomposite Design
Rev 1

Objective:

To verify that the proposed leachate collection system geocomposite drainage layer and pipe spacing is such that it maintains less than 1 foot of head on the liner under a conservative hydraulic scenario and has sufficient capacity to handle the anticipated peak leachate flow.

Procedure:

The Hydrologic Evaluation of Landfill Performance (HELP) Model and the Giroud Equation were utilized to evaluate the proposed leachate collection system. The proposed leachate collection system consists of a geocomposite drainage material, leachate collection piping at a maximum spacing of 200 feet, and a 12-inch layer of protective cover sand.

The HELP Model was utilized to determine the peak daily rate of leachate flow into the leachate collection system. A 3.6% floor grade was utilized in the model to represent the shallowest floor grade in the expansion design grades. (Note: Shallower grades may exist in isolated areas such as near the leachate collection sump. However, spacing between leachate collection pipes is significantly less in those areas).

Two scenarios were evaluated as part of this analysis:

- Open Conditions: This scenario used assumed a 10-ft lift of waste had been placed in the cell. The lift is modeled to have a 6 inch thick daily cover soil. This scenario provides a peak daily leachate flow under low load conditions, corresponding to a loading of approximately 900 psf assuming a waste density of 90 pcf.

- Intermediate Conditions: This scenario assumes 80-ft of waste has been placed in the cell. The waste material is assumed to be covered with 12-inches of intermediate cover soils, prior to the installation of the final closure cap. This scenario represents the maximum area and provides the maximum peak daily leachate flow corresponding to a maximum loading of approximately 7,200 psf.

The peak leachate liquid impingement rates calculated by the HELP Model were utilized to determine the required transmissivity of the geocomposite drainage material under both low load and high load conditions. The intent of the geocomposite design is to verify that the peak rate of leachate inflow can be contained entirely within the geocomposite drainage material. Therefore, the proposed 12-inch sand protective layer above the geocomposite is ignored, providing an added factor of safety to the design.

The Giroud Equation was utilized to calculate the minimum required transmissivity of the geocomposite based on the peak expected leachate flows from HELP, industry accepted reduction factors, and an added factor of safety of 2.0.

Additional reduction factors are added for each load based on conditions the geocomposite is predicted to encounter and the standards provided by Geosynthetic Research Institute standard GC-8.

The following table provides a summary of the reduction factors used in the calculations.

Geocomposite Reduction Factors		
Reduction Factor	**Open Condition**	**Intermediate Condition**
Elastic Deformation	1.0	1.5
Creep Deformation	1.5	2.0
Chemical Clogging	1.5	3.0
Biological Clogging	1.3	1.3
Root Penetration	1.0	1.0

Elastic deformation is typically not taken into account for the composite during the open conditions. The elastic deformations during these conditions are relatively negligible during low loads and a flat slope. According to current literature, the round shape of the solid core drainage net found in the composite material should not greatly affect the flow capacity. The deformation caused by the rotation of the net has a greater influence on the capacity of the drainage net component.

As such, the creep deformation is taken as 1.5 for the Open Condition as an interim creep as loading onto the composite is a general and conservative value for stresses below 6,000 psf. Loading under the intermediate condition with approximately 80 feet of material will generate a compressive stress of less than 7,200 psf, however the reduction factors for elastic and creep deformation are conservatively adjusted to 1.5 and 2.0, respectively.

Typical reduction for the chemical clogging of the geocomposite ranges from 1.5 to 3.0. Chemical clogging is taken into account for any reaction of the landfill material with that of the composite material. It is common for the design of leachate collection layers to assume a value of 1.5 however, maybe increased based on the alkalinity and particulate settlement of the material.

Biological Clogging is typically conservatively estimated at 1.1 to 1.3 based on Geosynthetic Research Institute GC-8 and 1.5 to 2.0 in more recent literature. However, this is typically a value used for municipal solid waste facilities with a larger potential for biological activities. A conservative value of 1.3 was considered in this analysis.

Results

Open Conditions

For the HELP Model analysis of open conditions, the maximum peak daily rate of leachate flow into the leachate collection system was found to be 0.16210 inches/day.

The geocomposite drainage material must have a minimum required transmissivity of 6×10^{-4} m^2/sec under low load conditions (900 psf) in order to convey the peak expected leachate flow during open conditions. Refer to calculation attachments for open condition HELP model and Giroud Equation calculations.

Intermediate Conditions

For the HELP Model analysis of intermediate conditions, the maximum peak daily rate of leachate flow into the leachate collection system was found to be 0.00323 inches/day.

The geocomposite drainage material must have a minimum required transmissivity of 4×10^{-4} m^2/sec under high load conditions (7,200psf) in order to convey the peak expected leachate flow under maximum loading conditions. Refer to calculation attachments for intermediate condition HELP model and Giroud Equation calculations.

Conclusions

The proposed leachate collection system can safely convey the peak expected leachate flows in the expansion area within the drainage geocomposite material, thus maintaining less than 1 foot of head on the liner. The minimum required transmissivities for the geocomposite drainage layer must be tested prior to construction to verify that it meets the minimum transmissivity values under high and low load conditions. The maximum spacing of the leachate laterals were calculated at 200 feet on center.

Tables from the GSE Design manual have been provided in referenced attachment. GSE geocomposite products may be evaluated by plotting the required transmissivity and flow gradient on each table. Based on the information, it appears that the GSE 250 mil FabriNet HF® and FabriNet TRx® are representative of acceptable materials for use in the project. However, transmissivity testing will be required to verify that the selected materials meet the transmissivities required by the results of this analysis. Refer to the project specifications and CQA Plan.

References

1. Giroud, J.P., J.G. Zornberg, and A. Zhao, 2000, "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers," *Geosynthetics International*, Special Issue on Liquid Collection Systems, Vol. 7, Nos. 4-6, pp. 285-380, pp. 453-489.
2. Li, Mengjia, and D. Narejo, 2010, "New round-strand innovation in biplanar geonet structure," *Geosynthetics*, Vol. 28, No. 2, pp. 19-27.

GEOCOMPOSITE CAPACITY ANALYSIS
Single Slope Scenario

LEACHATE LATERAL SPACING CALCULATION (10 FEET OF MATERIAL)

liquid impingement rate, q_h =	0.16210	in/day	*(Peak Daily Value collected above liner system - from HELP Model)*
	4.77E-06	cm/sec	

GEOCOMPOSITE PROPERTIES

ultimate transmissivity, Q =	6.0000E-04	m^2/sec	
aggregate thickness, t =	250	mil	Thickness of geonet
reduction factor for elastic deformation, RF_{in} =	1.0		Low load condition, neglible deformation of the geonet is expected to occur
reduction factor for creep deformation, RF_{cr} =	1.5		Low load condtion, interim creep deformation.
reduction factor for chemical clogging, RF_{cc} =	1.5		Based on GRI-GC8 recommendations for leachate collection systems.
reduction factor for biological clogging, RF_{bc} =	1.3		Conservative values selected based on GRI-GC-8
reduction factor for root penetration, RF_{rp} =	1.0		Liner material, no vegetative growth
design factor of safety, FS =	2.0		
allowable hydraulic conductivity, k =	1.6E+00	cm/sec	

SLOPE PROPERTIES

slope, z =	3.6	%	*(Min. floor slope)*
slope, b =	2.06	°	
length of slope, L =	200.000	ft	*(Flow length between leachate laterals)*

RESULTS

characteristic parameter, l =	2.3E-03	
modifying factor, j =	0.988242072	
max. liquid thickness, t_{max} =	0.20	in
max. liquid thickness, t_{max} =	196.82	mil

MAX. LIQUID THICKNESS:

$$\boxed{t_{\max} = \frac{q_h \ L}{k \ \sin \ \beta}}$$

** 196 mil is less than the thickness of the geocomposite and less then the maximum head (12-inches) allowed.*

WHERE:
t_{max} = maximum liquid thickness
q_h = liquid impingement rate
L = length of slope
k = hydraulic conductivity of geocomposite
b = slope

REFERENCE: Giroud, J.P., J.G. Zornberg, and J.F. Beech, 2000, "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers Comprising Two Different Slopes", *Geosynthetics International* , Special Issue on Liquid Collection Systems, Vol. 7, Nos. 4-6, pp 453-489.

GEOCOMPOSITE CAPACITY ANALYSIS
Single Slope Scenario

LEACHATE LATERAL SPACING CALCULATION (80 FEET OF MATERIAL)

liquid impingement rate, q_h =	0.00323	in/day	*(Peak Daily Value collected above liner system - from HELP Model)*
	9.50E-08	cm/sec	
GEOCOMPOSITE PROPERTIES			
ultimate transmissivity, Q =	4.0000E-04	m^2/sec	
aggregate thickness, t =	250	mil	Thickness of geonet
reduction factor for elastic deformation, RF_{in} =	1.5		High Load condition, however, neglible deformation of the geonet is expected to occur
reduction factor for creep deformation, RF_{cr} =	2.0		High load condtion, interim creep deformation.
reduction factor for chemical clogging, RF_{cc} =	3.0		Based on GRI-GC8 recommendations for leachate collection systems.
reduction factor for biological clogging, RF_{bc} =	1.3		Conservative values selected based on GRI-GC-8
reduction factor for root penetration, RF_{rp} =	1.0		Liner material, no vegetative growth
design factor of safety, FS =	2.0		
allowable hydraulic conductivity, k =	2.7E-01	cm/sec	
SLOPE PROPERTIES			
slope, z =	3.6	%	*(Min. floor slope)*
slope, b =	2.06	°	
length of slope, L =	200.000	ft	*(Flow length between leachate laterals)*
RESULTS			
characteristic parameter, l =	2.7E-04		
modifying factor, j =	0.998545329		
max. liquid thickness, t_{max} =	0.02	in	
max. liquid thickness, t_{max} =	23.53	mil	

MAX. LIQUID THICKNESS:

$$\boxed{t_{\max} = \frac{q_h \ L}{k \ \sin \ \beta}}$$

** 23 mil is less than the thickness of the geocomposite and less then the maximum head (12-inches) allowed.*

WHERE:
t_{max} = maximum liquid thickness
q_h = liquid impingement rate
L = length of slope
k = hydraulic conductivity of geocomposite
b = slope

REFERENCE: Giroud, J.P., J.G. Zornberg, and J.F. Beech, 2000, "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers Comprising Two Different Slopes", *Geosynthetics International*, Special Issue on Liquid Collection Systems, Vol. 7, Nos. 4-6, pp 453-489.

APPENDIX F.2D

Pipe Loading Calculations

I. PURPOSE

The purpose of this analysis is to estimate the stability of the proposed 6-inch diameter SDR-11 HDPE underdrain and leachate collection pipes for the proposed Scepter Inc. Waverly Class II Disposal Facility. The pipes were evaluated for wall crushing, wall buckling, and bending strain due to ring deflection failure mechanisms. The pipes were evaluated for the maximum expected loading produced over the life of the facility. Traffic loads during construction and post-closure load conditions were evaluated to determine the maximum stresses applied to the proposed HDPE pipes.

II. DESIGN ASSUMPTIONS AND CONDITIONS

The following assumptions were made for the pipe analysis:

- The leachate collection pipes are installed according to the design alignment, grade, and elevation;
- The "arching" of overburden overlying the pipes is conservatively neglected so the load from the total depth of overburden is applied to the pipe;
- As a non-rigid, non-pressurized pipe, the stability of the HDPE pipe is considered as part of a soil/pipe system;
- The total design period for the HDPE pipe is 50 years; and
- The maximum overburden above the HDPE pipe was estimated at 217 feet of waste material based on the final waste grades of the proposed facility.

III. MATERIAL CHARACTERISTICS

A 6-inch nominal diameter SDR-11 HDPE pipe is proposed for the underdrain to be constructed below the landfill liner. SDR-11 HDPE pipe is also proposed for the leachate collection system located at the base of the proposed landfill. The HDPE pipes have an outside diameter of 6.625 inches and a minimum wall thickness of 0.602 inches. The short-term modulus of elasticity is 110,000 psi and the long-term modulus of elasticity is 28,250 psi corresponding to an elastic modulus for HDPE after the 50 year design time at 73°F (from AWWA Manual M55, 2006). The bedding soil adjacent to all pipes is assumed to be TDOT No.57 stone.

IV. METHODOLOGY

The methodology utilized to calculate the pipe stability is from the American Water Works Association (AWWA) Manual M55, PE Pipe – Design and Installation, 2006 (referred to herein as AWWA Manual). HDPE pipes will be estimated for resistance against pipe wall crushing, pipe wall buckling, and excessive bending strain due to ring deflection under the maximum vertical loads anticipated at the landfill. Reasonably conservative assumptions were made for the factors affecting each analysis.

a. Vertical Pressure

Pressures applied to the proposed HDPE piping was estimated for two load conditions: (i) construction traffic loading and (ii) post-closure loading. The maximum vertical pressure from the two loading conditions was used to evaluate the pipe stability.

(i) Construction Traffic Loading

The anticipated construction traffic consists of a live load applied by a truck with a total wheel load of 40,000 lb when full onto a pipe with minimal cover. For this calculation, it is assumed that the pipe will have a minimum of two feet of TDOT No.57 stone cover for both the SDR 11 HDPE underdrain and leachate collection piping. The total stress on the pipe is the sum of the stresses applied by the soil cover and the stresses applied by the loaded truck. The total pressure is calculated using the following equation (derived from the AWWA Manual, equation 5-2).

$$P_L = \frac{\gamma_p H_c}{144} + \frac{I_f W_L}{A_c} \cdot \left[1 - \frac{{H_c}^3}{\left({R_{eq}}^2 + {H_c}^2\right)^{1.5}}\right]$$

Where:

P_L = Vertical pressure on pipe due to construction traffic, psi

γ_p = In-place density of cover material, pcf

H_c = Soil height above pipe crown, ft

I_f = Impact factor

W_L = Wheel load, lbs

A_c = Tire contact area, in^2

R_{eq} = Equivalent radius, ft, given by $R_{eq} = \sqrt{\frac{A_c}{\pi}}$

(ii) Post-Closure Loading

Following closure, the pipe will be loaded by the added surcharge from soil/waste on the pipes. The vertical surcharge pressure from the waste was estimated in the settlement analysis (see Appendix F.4F Settlement Analysis) using an embankment loading procedure. The maximum surcharge pressure was calculated to be 21,965 psf (152.53 psi).

Based on the calculations for traffic loading and post-closure loading, the maximum vertical pressure that is anticipated to load the pipes is from the post-closure loading. The vertical pressure from the post-closure loading was used to evaluate the pipe stability.

b. Wall Crushing

Pipe wall crushing occurs when stress in the pipe wall exceeds the compressive strength of the pipe material. The compressive stress on the pipe wall is calculated using the following equation (AWWA Manual, equation 5-15).

$$\sigma_c = \frac{P_{max}(SDR-1)}{2}$$

Where:

σ_c = Pipe wall compressive stress, psi

P_{max} = Maximum vertical pressure on pipe, psi

SDR = Standard dimension ratio of HDPE pipe

The compressive stress applied must be less than the allowable compressive stress provided by the pipe manufacturer.

c. Wall Buckling

The potential for long-term pipe wall buckling failure, a longitudinal wrinkling in the pipe wall, is dependent upon the constrained pipe/soil system's critical buckling pressure. The critical wall buckling pressure is calculated using the following equation (AWWA Manual, equation 5-10).

$$P_{wb} = 5.65\sqrt{R_b \cdot B' \cdot E' \cdot \frac{E_{pipe}}{12(SDR-1)^3}}$$

Where:

P_{wb} = Critical constrained pipe wall buckling pressure, psi

E' = Modulus of soil reaction for pipe bedding, psi

E_{pipe} = Long-term elastic modulus of pipe material, psi

H_f = Final cover above pipe crown, ft

H_w = Groundwater height above pipe, ft

R_b = Buoyancy reduction factor, $R_b = 1 - (0.33\frac{H_w}{H_f})$

B' = Elastic support factor, $B' = \frac{1}{1+4e^{-0.065 \cdot H_f}}$

The factor of safety against pipe wall buckling is calculated using the following equation.

$$FS_{wb} = \frac{P_{wb}}{P_{max}}$$

Where:

FS_{wb} = Safety factor against pipe wall buckling

The factor of safety against pipe wall buckling must be greater than or equal to two.

d. Ring Deflection

Ring deflection causes the pipe to deflect elliptically which can lead to excessive bending strain in the pipe long term and reduce flow capacity. Ring deflection is calculated using Spangler's Modified Iowa Formula (AWWA Manual, equation 5-8).

$$\Delta X\% = \frac{\Delta X}{D_i} \cdot 100 = \frac{P_{max} K_b L_d}{\frac{2E_{pipe\_s}}{3(SDR-1)^3} + 0.061 \cdot E'}$$

Where:

$\Delta X\%$ = Percent horizontal ring deflection

ΔX = Horizontal ring deflection

K_b = Bedding constant, typically 0.1

L_d = Time-lag factor

E_{pipe_s} = Short-term elastic modulus of pipe material, psi

For non-pressurized pipe applications, a 7.5% deflection limit provides a large safety factor against bending strain and is considered a safe design deflection.

V. RESULTS

The calculations are provided in Attachment A. The following table summarizes the results from the calculations.

Table 1. Pipe Loading Analysis Results

Load Condition	Acceptance Criteria	Calculated
Wall Crushing	applied compressive stress must be less than or equal to allowable pipe compressive strength (1000 psi)	763 psi < 1000 psi
Wall Buckling	FS ≥ 2.0	FS = 3.1 > 2.0
Ring Deflection	percent ring deflection must be less than or equal to 7.5%	7.4% < 7.5%

The results of the analysis indicate that the proposed HDPE pipes satisfy the requirements specified in the AWWA Manual.

VI. CONCLUSIONS

The 6-inch diameter HDPE SDR-11 leachate collection pipe and underdrain piping is stable under the maximum vertical stress applied from the waste embankment during post-closure conditions. The pipes exhibit acceptable capacity against pipe wall crushing, pipe wall buckling, and bending strain due to ring deflection. Construction traffic loading is not anticipated to damage the piping when a minimum of two feet of soil cover is provided.

VII. REFERENCES

AWWA Manual M55, PE Pipe – Design and Installation, 2006.

AECOM, Appendix F.4F Settlement Analysis.

VIII. ATTACHMENTS

Attachment A Detailed Calculations

Attachment A

Reference AWWA Manual M55, PE Pipe - Design and Installation, Chapter 5

Outside pipe diameter $D_o := 6.625\text{in}$ (6" SDR 11 pipe)

Minimum pipe thickness $t_{pipe} := 0.602\text{in}$

Inner pipe diameter $D_i := D_o - 2 \cdot t_{pipe} = 5.421 \cdot \text{in}$

Standard dimension ratio of pipe $SDR := 11$

In-place density of cover material $\gamma_p := 150\text{pcf}$ (No.57 stone)

Vertical depth of embedment to pipe crown $H_c := 2\text{ft}$ (Temporary condition)

Vertical depth of embedment to pipe crown $H_f := 217\text{ft}$ (Final condition)

Wheel load $W_L := 40000\text{lbf}$

Contact area $A_c := 10\text{in} \cdot 20\text{in} = 200 \cdot \text{in}^2$

Equivalent radius $R_{eq} := \sqrt{\frac{A_c}{\pi}} = 0.665 \cdot \text{ft}$

Impact factor (assumed 2.5 for unpaved roads) $I_f := 2.5$

Pressure applied from truck traffic $P_L := \gamma_p \cdot H_c + \frac{I_f \cdot W_L}{A_c} \cdot \left[1 - \frac{H_c^{\,3}}{\left(R_{eq}^{\,2} + H_c^{\,2}\right)^{1.5}}\right] = 74.837\,\text{psi}$

Maximum surcharge load $P_t := 21965\text{psf} = 152.53\,\text{psi}$ (Post-closure loading)
(obtained from settlement analysis)

Maximum pressure from embankment loading and truck traffic $P_{max} := \max\left(P_t, P_L\right) = 152.535\,\text{psi}$

WALL CRUSHING

Pipe wall compressive stress $\sigma_c := \dfrac{P_{max}\cdot(SDR-1)}{2} = 762.674\cdot psi$

Allowable compressive yield strength of pipe $\sigma_y := 1000psi$

Check allowable compressive strength of pipe $check_{wc} := \begin{cases} \text{"Satisfactory"} & \text{if } \sigma_c \le \sigma_y \\ \text{"No good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$

WALL BUCKLING

Modulus of soil reaction for pipe bedding $E' := 3000psi$

Long-term elastic modulus of pipe $E_{pipe} := 28250psi$ (at 73°F)

Groundwater height above pipe $H_w := 0ft$

Buoyancy reduction factor $R_b := 1 - \left(0.33\cdot\dfrac{H_w}{H_f}\right) = 1$

Elastic support factor $B' := \dfrac{1}{1 + 4\cdot e^{\frac{-0.065}{ft}\cdot H_f}} = 1$

Critical constrained wall buckling pressure $P_{wb} := 5.65\cdot\sqrt{R_b\cdot B'\cdot E'\cdot\dfrac{E_{pipe}}{12\cdot(SDR-1)^3}} = 474.818\,psi$

Safety factor against wall buckling $FS_{wb} := \dfrac{P_{wb}}{P_{max}} = 3.113$

Check safety factor against wall buckling $check_{wb} := \begin{cases} \text{"Satisfactory"} & \text{if } FS_{wb} \ge 2.0 \\ \text{"No good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$

BENDING STRAIN DUE TO RING DEFLECTION

Bedding constant $K_b := 0.1$

Deflection lag factor $L_d := 1.25$

Short-term elastic modulus of pipe $E_{pipe\_s} := 110000psi$ (at 73°F)

Percent ring deflection

$$\Delta X\% := \frac{P_{max} \cdot K_b \cdot L_d}{\frac{2 \cdot E_{pipe\_s}}{3 \cdot (SDR - 1)^3} + 0.061 \cdot E'} = 7.438 \cdot \%$$

Check ring deflection

$$check_{\Delta X\%} := \begin{cases} \text{"Satisfactory"} & \text{if } \Delta X\% \le 7.5\% \\ \text{"No good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$$

For non-pressurized pipes, a 7.5% deflection limit provides a large safety factor against instability. The percent ring deflection is less than the deflection limit.

APPENDIX F.2E

Protective Cover Graded Filter Evaluation

AECOM

Job	Scepter Waverly Landfill	Project No.	60398526	Sheet	1 Of 12
Description	Protective Cover Filter Calculation	Computed by	MW	Date	6/28/16
		Checked by	MWB	Date	6/30/16

I. PURPOSE

The purpose of this calculation is to select acceptable particle size distribution bands for the proposed protective cover sand layer of the geosynthetic liner system of the Scepter Waverly Class II Disposal Facility, located in Waverly, Tennessee. The proposed protective cover sand will also be evaluated to determine acceptable geotextile apparent opening size properties for the proposed leachate collection drainage system.

II. SITE AND PROJECT DESCRIPTION

The graded filter calculations have been performed as part of the industrial waste permit application for the proposed landfill at the Scepter Site, located in Waverly, Tennessee. The proposed landfill will be permitted as a new Class II industrial solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management.

The geosynthetic liner system for the proposed Class II industrial solid waste landfill will consist of the following components: Immediately below the waste (which will consist of Aluminum Salt Cake), a 1-foot thickness of protective cover soil will be constructed using on-site cohesive materials excavated during the mass grading for the landfill footprint. A 1-foot thickness of protective cover sand will be constructed immediately below the protective cover soil layer. Geosynthetics including a geocomposite drainage layer, flexible membrane liner, and geosynthetic clay liner will be placed below the protective sand cover layer. A 2-foot thickness of compacted soil will be constructed below the geosynthetic layer using on-site cohesive materials excavated during the mass grading for the landfill footprint. The proposed liner system is illustrated below in Figure 1.

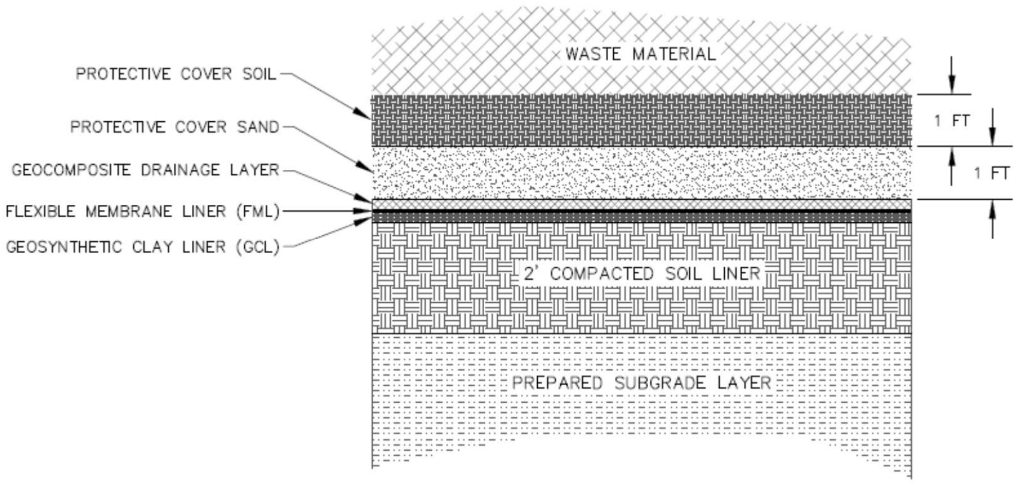


Figure 1: Proposed Landfill Liner System

For the proposed leachate collection system, protective sand cover will be placed over an 8 ounce per square yard (oz./sy.) non-woven geotextile fabric that will envelop AASHTO #57 stone and a 6 inch diameter, perforated SDR-11 high density polyethylene (HDPE) collection pipe. Because the geotextile will be overlain by the protective cover sand, the geotextile must be sized to adequately to filter the protective cover sand material. The proposed leachate collection system is shown below in Figure 2.

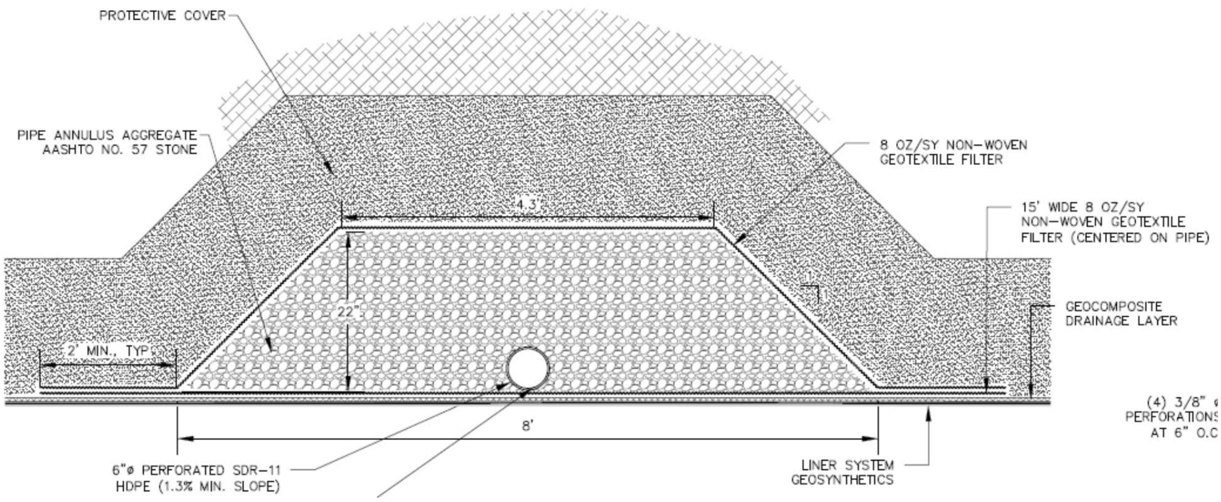


Figure 2: Proposed Leachate Collection System

The following sections summarize the methodology, assumptions, and results of the graded filter design and geotextile apparent opening size calculations. For further detail on the specific calculations performed, refer to the corresponding data provided in Attachment A.

III. GRADED FILTER CALCULATION – METHODOLOGY

Filter gradation limits are determined using the following steps, as recommended in the NRCS National Engineering Handbook, Part 633, Chapter 26: Gradation Design of Sand and Gravel Filters (Category 2).

a. Determine the grain-size distribution of the base soil material (cohesive cover soil). Use enough samples to define the range of grain sizes for the base soil or soils and design the filter gradation based on the base soil that requires the smallest D_{15} size.

b. Prepare adjusted gradation curves for base soils that have particles larger than the No. 4 (4.75 mm) sieve.

   1) Obtain a correction factor by dividing 100 by the percent passing the No. 4 sieve.

2) Multiply the percentage passing each sieve size of the base soil smaller than No. 4 sieve by the correction factor determined above.
3) Plot these adjusted percentages to obtain a new gradation curve.
4) Use the adjusted curve to determine the percentage passing the No. 200 (0.075 mm) sieve in part c below.

c. Place the base soil in a category determined by the percent passing the No. 200 sieve from the regraded gradation curve data according to Table 26-1.

Table 26–1 Regraded gradation curve data

Base soil category	% finer than No. 200 sieve (0.075 mm) (after regrading, where applicable)	Base soil description
1	> 85	Fine silt and clays
2	40 – 85	Sands, silts, clays, and silty & clayey sands
3	15 – 39	Silty & clayey sands and gravel
4	< 15	Sands and gravel

d. To satisfy filtration requirements, determine the maximum and minimum allowable D_{15} size for the filter in accordance with Tables 26-2 and 26-3, respectively.

Table 26–2 Filtering criteria — Maximum D_{15}

Base soil category	Filtering criteria
1	$\leq 9 \times d_{85}$ but not less than 0.2 mm
2	≤ 0.7 mm
3	$\leq \left(\frac{40-A}{40-15}\right)\left[\left(4\times d_{85}\right)-0.7\text{mm}\right]+0.7\text{mm}$ A = % passing #200 sieve after regrading (If 4 x d_{85} is less than 0.7 mm, use 0.7 mm)
4	$\leq 4 \times d_{85}$ of base soil after regrading

Table 26–3 Permeability criteria

Base soil category	Minimum D_{15}
All categories	$\geq 4 \times d_{15}$ of the base soil before regrading, but not less than 0.1 mm

e. The width of the allowable filter design band must be kept relatively narrow to prevent the use of possibly gap-graded filters. Adjust the maximum and minimum D_{15} sizes for the filter band determined in step d. so that the ratio is 5 or less at any given percentage passing of 60 or less. Criteria are summarized in Table 26-4.

In designing an acceptable filter band using the preliminary control points obtained in steps a. through d., the following additional requirements should be followed to decrease the probability of using a gap-graded filter.

1. Locate the band at the minimum D_{15} side of the range if it is probable that there are finer base materials than those sampled and filtering is the most important function of the zone. With the minimum D_{15} size as the control point, establish a new maximum D_{15} size by multiplying the minimum D_{15} size by 5, and locate a new maximum D_{15} size.
2. The most important consideration may be to locate the maximum and minimum D_{15} sizes, within the acceptable range of sizes determined in step d., so that a standard gradation available from a commercial source or other gradations from a natural source near the site would fall within the limits. Locate a new maximum D_{15} and minimum D_{15} within the permissible range to coincide with the readily available material. Ensure that the ratio of these sizes is 5 or less. Label the maximum D_{15} size Control point 1 and the minimum D_{15} size Control point 2.

f. Adjust the limits of the design filter band so that the coarse and fine sides have a coefficient of uniformity (CU) of 6 or less by using the following equation:

$$CU = \frac{D_{60}}{D_{10}} \leq 6$$

Initial design filter bands by this step will have CU values of 6. For final design, filter bands may be adjusted to a steeper configuration, with CU values less than 6, if needed. This is acceptable so long as other filter and permeability criteria are satisfied.

Calculate a maximum D_{10} value equal to the maximum D_{15} size divided by 1.2. The factor of 1.2 is based on the assumption that the slope of the line connecting D_{15} and D_{10} should be on a coefficient of uniformity of approximately 6. Calculate the maximum permissible D_{60} size by multiplying the maximum D_{10} value by 6. Label this Control point 3.

Determine the minimum allowable D_{60} size for the fine side of the band by dividing the determined maximum D_{60} size by 5. Label this Control point 4.

Table 26–4 Other filter design criteria

Design element	Criteria
To prevent gap-graded filters	The width of the designed filter band should be such that the ratio of the maximum diameter to the minimum diameter at any given percent passing value ≤ 60% is ≤ 5.
Filter band limits	Coarse and fine limits of a filter band should each have a coefficient of uniformity of 6 or less.

g. Determine the minimum D_5 and maximum D_{100} sizes of the filter according to Table 26-5. Label as Control points 5 and 6, respectively.

Table 26–5 Maximum and minimum particle size criteria*

Base soil category	Maximum D_{100}	Minimum D_5, mm
All categories	≤ 3 inches (75 mm)	0.075 mm (No. 200 sieve)

* The minus No. 40 (.425 mm) material for all filters must be nonplastic as determined in accordance with ASTM D4318.

h. To minimize segregation during construction, the relationship between the maximum D_{90} and minimum D_{10} sizes of the filter is important. Calculate a preliminary minimum D_{10} size by dividing the minimum D_{15} size by 1.2. Determine the maximum D_{90} using Table 26-6. Label this as Control point 7.

Table 26–6 Segregation criteria

Base soil category	If D_{10} is :	Then maximum D_{90} is:
	(mm)	(mm)
All categories	< 0.5	20
	0.5 – 1.0	25
	1.0 – 2.0	30
	2.0 – 5.0	40
	5.0 – 10	50
	> 10	60

i. Connect Control points 4, 2, and 5 with straight lines to form a partial design for the fine side of the filter band. Connect Control points 6, 7, 3, and 1 with straight lines to form a design for the coarse side of the filter band. This results in a preliminary design for a filter band. Complete the design by extrapolating the coarse and fine curves to the 100 percent finer value. For the purposes of writing specifications, select appropriate sieves and corresponding percent finer values that best reconstruct the design band and tabulate the values.

IV. GEOTEXTILE FILTER CALCULATION – METHODOLOGY

The Apparent Opening Size (AOS, or O_{95}) of the geotextile represents approximately the largest soil particle size that will pass through a given geotextile. The following relationships have been developed in order to determine whether a given geotextile is capable of filtering or retaining the soil upstream of the geotextile depending on the gradation of the upstream soil:

$For\ soil\ \leq 50\%\ passing\ the\ No.200\ sieve:$

$$O_{95,geotextile} < 0.60mm\ (AOS\ \geq No.30\ sieve)$$

$For\ soil\ \geq 50\%\ passing\ the\ No.200\ sieve:$

$$O_{95,geotextile} < 0.30mm\ (AOS\ \geq No.50\ sieve)$$

Another widely used design criteria recommends the following relationship be met (Carroll, 1983):

$$O_{95,geotextile} < 2\ to\ 3\ \times D_{85,base}$$

Therefore, a $O_{95,geotextile}$ will be determined for the proposed geotextile fabric such that each of the above criteria are met.

V. GRADED FILTER MATERIAL PROPERTIES

As discussed, the primary purpose of the graded filter analysis is to establish an acceptable filter band for the protective cover sand that will serve to filter the protective cover soil, consisting of on-site cohesive material excavated during the mass grading of the landfill. In order to assess the properties of the protective cover soil, individual particle size distribution tests (ASTM D 422) were performed on protective cover soil samples taken from multiple borings located across the proposed landfill footprint. These borings were collected as part of the recent Hydrogeologic Exploration. In total, 21 individual tests were assessed, and the gradations were fully corrected for gravel content as described in Section III a. and b. The corrected, particle size distribution plots are shown below in Figure 3.

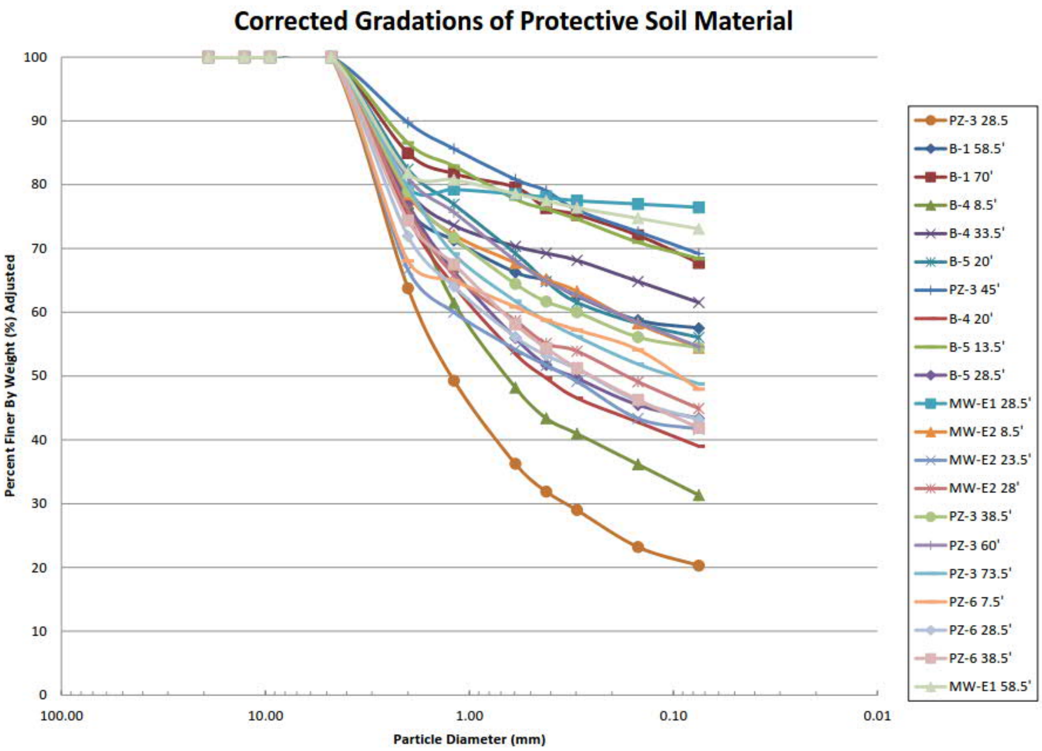


Figure 3: Corrected particle size distribution for Protective Cover Soil

As shown by the results presented above in Figure 2, the vast majority of the corrected gradations have between 40 and 85 percent fine sized particles (finer than 0.075 mm, or the opening of the Number 200 mesh sieve). Specifically, 46 particle size distribution tests were performed as part of the Hydrogeological Exploration, and by analysis, 43 of 46 tests (or 93%) demonstrated between 40 and 85 percent fine sized particles. On average, for every sample tested, 52% of the particles are fine sized after correcting for gravel content. On this basis, the protective cover soil that will be constructed of on-site cohesive materials is assessed as a Category 2 base soil in accordance with Section III, c. and Table 26-1.

VI. GRADED FILTER AND GEOTEXTILE CALCULATION RESULTS

Following the procedure described in Section III, the control points corresponding to the required filter bands for the protective sand cover were calculated based on the protective soil layer being a Category 2 base. The results of the analysis are summarized below in Table 1. Detailed calculations have also been provided in Attachment A to this document.

Table 1: Summary of Control Point Calculation

Control Point	Filter Band Designation	Calculated Particle Size (mm)	Basis
Control Point 1	D_{15MAX}	0.50[1]	$D_{15MAX}/D_{15MIN} = 5$
Control Point 2	D_{15MIN}	0.10	Minimum for Category 2 Base Materials
Control Point 3	D_{60MAX}	2.50	$D_{60MAX}/D_{10MAX} = 6$ ($C_u = 6$)
Control Point 4	D_{60MIN}	0.50	$D_{60MAX}/D_{60MIN} = 5$
Control Point 5	D_{5MIN}	0.075	Table 26-5
Control Point 6	D_{100MAX}	75	Table 26-5
Control Point 7	D_{90MAX}	20	Table 26-6 ($D_{10MIN} = 0.083$mm)
	D_{10MAX}	0.42	$D_{10MAX} = D_{15MAX}/1.2$
	D_{10MIN}	0.083	$D_{10MIN} = D_{15MIN}/1.2$

[1] The D_{15MAX} corresponding to a Category 2 Base material is ≤ 0.7mm. The ratio of D_{15MAX}/D_{15MIN} should be no greater than 5. As filtration is the most critical aspect of the filter design, it was determined that D_{15MIN} should control the filter band width at the 15th percentile.

Once the control points were determined, the filter design was completed by extrapolating the coarse and fine curves of the filter band to the 100 percent finer value. The completed filter band design requirements are summarized below in Table 2. Extrapolated values are noted.

Table 2: Summary of Filter Band Design

% Passing	Sieve Size (mm)	
	Minimum Filter Band	Maximum Filter Band
100	1.4[2]	75
90	1.1[2]	20
60	0.5	2.5
15	0.1	0.5
10	0.083	0.42
5	0.075	0.35[2]
0	--	--

[2] Extrapolated value

In addition, the proposed filter design band is shown graphically below in Figure 4.

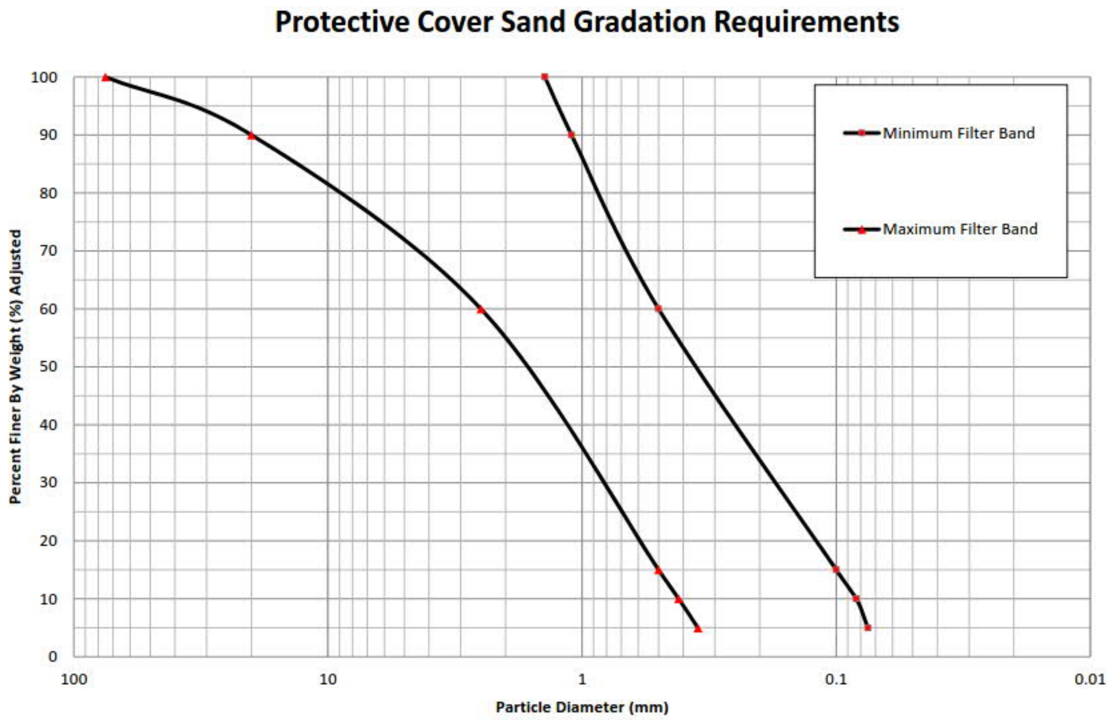


Figure 4: Filter Band Design Requirements

For the Non-woven Geotextile material proposed for the leachate collection system, the analysis indicates that the Apparent Opening Size (AOS, or O_{95}) of the proposed geotextile must meet the following criteria to adequately filter the proposed cover sand.

Since < 50% of the protective cover sand passes the number 200 sieve,

$$O_{95,geotextile} \ < 0.60mm \ (AOS \ \geq No. 30 \ sieve)$$

The D_{85} for the required protective cover sand will vary from 1mm to 15mm. Therefore, the AOS should also meet the following criteria:

$$O_{95,geotextile} \ < 2 \ to \ 3 \ \times D_{85=} \ < 2 \ to \ 3 \ mm$$

VII. CONCLUSIONS

In conclusion, it is recommended that the protective cover sand layer have a gradation that fits within the bounds of the filter band shown above in Figure 4. ASTM C33 Concrete Sand is a widely used, commercial grade material that is commonly used as a graded filter and which meets the filter band requirements for this project. The gradation of ASTM C33 Concrete Sand is presented below in Table 3 for reference. In addition, the gradation of the C33 Concrete Sand is shown in Figure 5 to be within the bands of the filter design. In any case, it should be stated that any material that meets the filter gradation criteria provided in Figure 4 and Table 2 is acceptable for use as protective cover sand.

Table 3: Gradation Requirements for ASTM C33 Concrete Sand

Sieve or Mesh	Size (mm)	% Passing	
		Minimum	Maximum
3/4"	19.05	100	100
1/2"	12.7	100	100
3/8"	9.525	100	100
#4	4.76	95	100
#8	2.38	80	100
#16	1.19	50	85
#30	0.59	25	60
#50	0.297	10	30
#100	0.149	2	10
#200	0.075	3	5

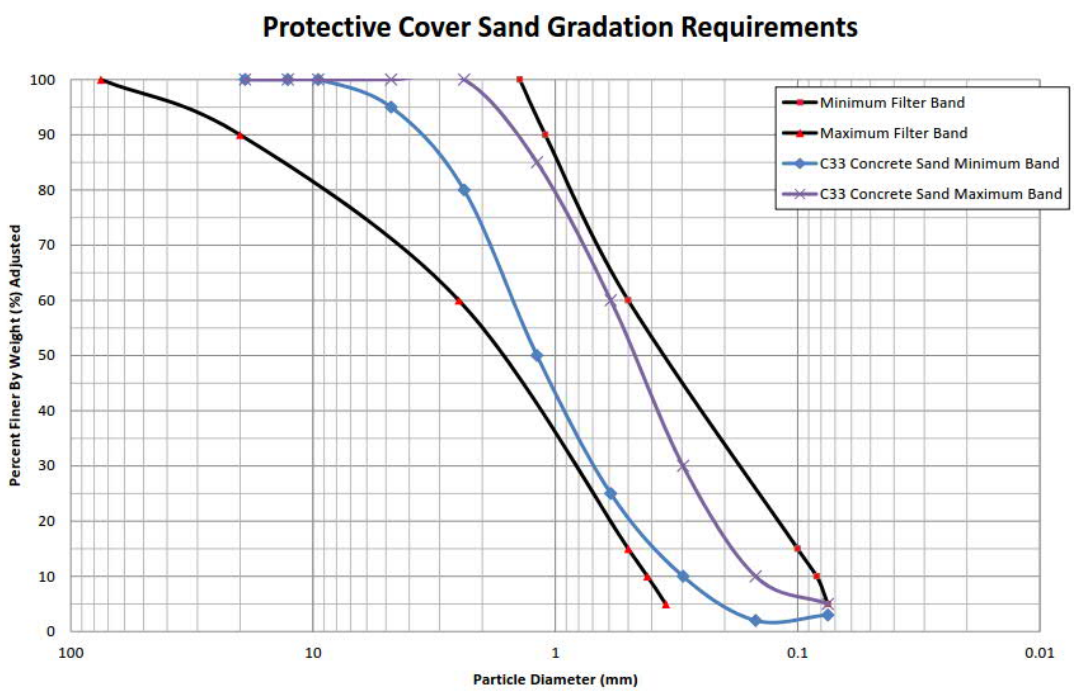


Figure 5: Comparison of ASTM C33 Concrete Sand with Required Filter Band Limits

VIII. REFERENCES

Carrol, R.G., Jr., Geotextile Filter Criteria, TRR 916, Engineering Fabrics in Transportation Construction, Washington DC, 1983, pp.46-53

Natural Resources Conservation Service (NRCS), 2013 (b). Part 633 National Engineering Handbook. United States Department of Agriculture. Chapter 26 – Gradation Design of Sand and Gravel Filters.

Report on Task Force #25, Joint Committee Report of AASHTO-AGC-ARTBA, American Association of State, Highway, and Transportation Officials, Washington DC, January 1991.

IX. ATTACHMENTS

Attachment A Detailed Filter Analysis Calculations

Attachment A

AECOM

Job SCEPTER WAVERLY LANDFILL | Project No. 60398526 | Page 1 of 1
Description FILTER DESIGN - CHAPTER 26 NRCS DESIGN MANUAL | Computed by (MW) | Sheet ___ of ___ | Date ___
Checked by ___ | Date ___

Reference

PROTECTIVE COVER SAND FILTER DESIGN

A. BASED ON CORRECTED GRADATIONS FROM ASTM D-422 TESTS, THE BASE MATERIAL HAS A % PASSING #200 SIEVE OF 40 TO 85. THEREFORE, CATEGORY 2. BASE.

B. FROM TABLE 26-2, $D_{15\,max} = 0.7mm$

FROM TABLE 26-3, $D_{15\,min} = 0.1mm.$

$\frac{D_{15\,max}}{D_{15\,min}} \leq 5.$ ∴ BECAUSE FILTERING IS THE MAIN FUNCTION, THE $D_{15\,min}$ WILL CONTROL.

THEREFORE, $D_{15\,max} = 0.5mm$
$D_{15\,min} = 0.1mm.$

CONTROL POINT 1 = 0.5mm.
CONTROL POINT 2 = 0.1mm

C. $D_{10\,max} = D_{15\,max} / 1.2 = 0.5mm / 1.2 = \underline{0.42mm}$

D. $D_{60\,max} = D_{10\,max} \cdot 6 = 0.42mm \times 6 = \underline{2.50mm}$
CONTROL POINT 3 = 2.50mm

$D_{60\,min} = D_{60\,max} / 5 = 2.50mm / 5 = \underline{0.50mm}$
CONTROL POINT 4 = 0.50mm

E. $D_{5\,min} = 0.075mm$ CONTROL POINT 5 = 0.075mm
$D_{100\,max} = 75mm$ CONTROL POINT 6 = 75mm

F. FROM TABLE 26-6, $D_{90\,max} = 20mm$ CONTROL POINT 7 = 20mm

$D_{10\,min} = D_{15\,min} / 1.2 = 0.1mm / 1.2 = \underline{0.083mm}$

APPENDIX F.2F

Leachate Secondary Containment

AECOM

Job Scepter: East Landfill	Project No. 60398526	Page 1 of 1	
Description Leachate Secondary Containment Volume Calculations	Computed by N. Popkowski	Sheet ___ of ___	Date 5/19/17
	Checked by M. May	Date 9/21/17	

Reference

① Determine Primary Containment Volume

Volume = Number of Tanks × Tank Volume

Tank Volume = 10,000 gallons
Number of Tanks = 3

Volume = 3 × 10,000 gal
= 30,000 gal ← Primary Storage Volume

② Determine Required Secondary Containment Volume

V_{req} = Primary Volume + 10% = Primary Volume × 1.10

V_{req} = 30,000 gal × 1.1
= 33,000 gal ← Required Secondary Containment Volume

③ Determine Actual Secondary Containment Volume

V_{act} = Length × Width × Height

wall thickness

Length = 55.5' − 8/12' − 8/12' = 54.17 feet
Width = 24.0' − 8/12' − 8/12' = 22.67 feet
Height = 3.6'

V_{act} = 54.17' × 22.67' × 3.6'
= 4,420 cf = 33,064 gal ← Actual Secondary Containment

④ Secondary Containment Summary

Actual Secondary Containment ≥ Required Secondary Containment

33,064 gal > 33,000 gal

AECOM

Job Scepter : EAST LandFill | Project No. 60398526 | Page ____ of ____ | Sheet ____ of ____

Description Pre-Existing Vertical Tanks Secondary Containment Volume calculations

Computed by M. May | Date 4/5/18

Checked by N. Poplowski | Date 4/5/18

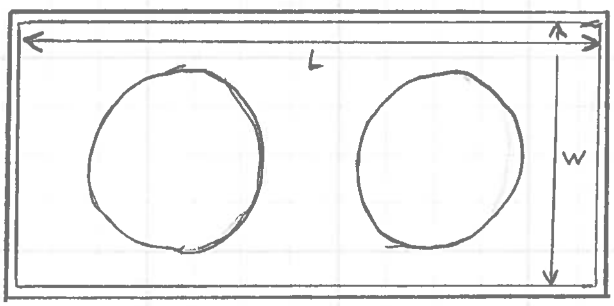
Reference

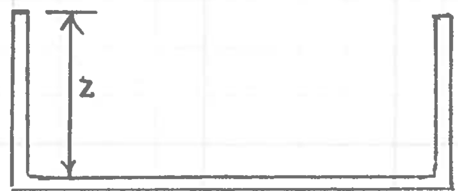
① Primary Volume = 2 tanks x 10,000 Gal each = 20,000 Gal

② Required Secondary Containment Volume (V_{Req})

V_{Req} = 20,000 Gal x 1.10 = 22,000 Gal

Note: Tanks connected on outlet with common manifold piping





L = 34' ½" = 34.04'

W = 22' 2" = 22.17'

Z = 42" = 3.5'

* Dimensions Field measured by Scepter personnel

③ Actual Secondary Containment Volume

$34.04' \times 22.17' \times 3.5' = 2{,}642\ Ft^3$

$2{,}642\ Ft^3 \times 7.48\ \frac{Gal}{Ft^3} = 19{,}762\ Gal$

$V_{ACT} < V_{Req}$

19,762 Gal 22,000 Gal

④ Modification Needed to obtain V_{Req}

$V_{Req} - V_{Act} = 22{,}000\ Gal - 19{,}762\ Gal = 2{,}242\ Gal$

$\Rightarrow 2{,}242\ Gal \times \frac{1\ Ft^3}{7.48\ GAL} = 299.7\ Ft^3$

Raise Wall height to create additional 299.7 Ft^3 capacity

Basin Area = $34.04' \times 22.17' = 754.6\ Ft^2$

$\Rightarrow \frac{299.7\ Ft^3}{754.6\ Ft^2} = 0.39\ Ft = 4.68\ in$

AECOM

Page ______ of ______
Sheet ______ of ______

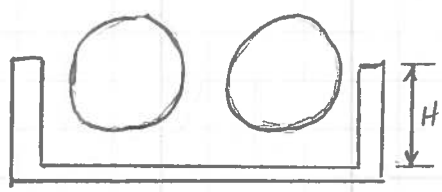
Job: Scepter : EAST LandFill
Project No.: 60398526
Description: Pre existing Horizontal Tanks Secondary Containment Volume Calculations
Computed by: M. May   Date: 4/5/18
Checked by: N. Popkowski   Date: 4/5/18
Reference

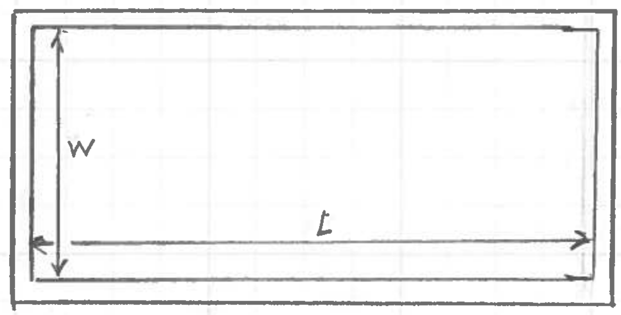
① Primary VOLUME = 2 tanks x 10,000 Gal each = 20,000 Gal

② Required Secondary Containment Volume (V_{Req})

V_{Req} = 20,000 Gal x 1.10 = 22,000 Gal

Note: Tanks connected on outlet with common manifold piping





L = 35′ 3½″ = 35.29′
W = 27′ 9″ = 27.75′
H = 40″ = 3.33′

* Dimensions Field Measured by Scepter personnel

③ Actual Secondary Containment Volume (V_{Act})

$35.29' \times 27.75' \times 3.33' = 3{,}261\ Ft^3$

$3{,}261\ Ft^3 \times 7.48\ \frac{Gal}{Ft^3} = 24{,}392\ Gal$

V_{ACT} > V_{Req}
24,392 Gal > 22,000 Gal

Scepter Landfill - East Phase

Leachate Generation Calculations

Stage of Facility Development	Area in Open Condition (acres)	Area in Intermediate Condition (acres)	Area in Pre-closure Condition (acres)	Average 30 day Leachate Generation Rate (Gallons)
Phase 1 Development	0.5	6.5	1.5	36,158
Phase 2 Development	0.5	7	8.6	67,030
Phase 3 Development	0.5	5	4.5	48,514
Phae 4 Development	0.5	5	4.5	48,514

HELP Model Results

	Highest Monthly Avg. (inches)	Daily Avg. (gal/acre/day)
Open Condition	2.06	1804.320774
Intermediate Condition	0.0153	13.40102323
Pre-Closure Condition	0.1644	143.9953084

	Active Area* (Acres)
Phase 1 Development	7.5
Phase 2 Development	16.1
Phase 3 Development	10
Phae 4 Development	10

* Active area is all areas inside the waste limits that has not been closed

April 18, 2018

Brian Griffin
Scepter Inc
Waverly, TN

Email: bgriffin@scepterinc.com

Regarding: Non-Regulated Leachate Disposal

Based on our experience with Hoover Mason's, Smelter Services' and Athens Hocking's landfill leachate, Valicor Environmental Services will be a disposal option for Scepter. Analytical will need to be reviewed by our compliance team prior to approval. Assuming the leachate is similar to those already being disposed by Valicor, the leachate would be approved for disposal at our Huntsville, Alabama facility.

Should you have any other questions, please let us know.

Thank you.

Sean Westmeier
Valicor Environmental Services
Cell: 513-615-6777
Alabama Facility: 256-851-5554

APPENDIX F.3

Final Cover System Calculations

F.3A Alternative Cover System Demonstration

F.3B Final Cover System Geocomposite Design

APPENDIX F.3A

Alternative Cover System Demonstration

AECOM

Job	Scepter Waverly Landfill East Phase Site	Project No.	60398526	Sheet	1 of 5
Description	Alternative Final Cover System	Computed by	YRC	Date	06/30/16
	Demonstration	Checked by	NSG	Date	06/30/16

I. PURPOSE

The objective of this analysis is to demonstrate that the Geosynthetic final cover system provides equivalent or superior performance when compared to the compacted clay final cover system. The following analysis was performed using the U.S. Environmental Protection Agency's (USEPA) Hydrologic Evaluation of Landfill Performance (HELP) Model, version 3.07, to provide a comparison of the amount of leakage generated through the two cover systems. The results of the HELP Model provide a long term estimation of water migration into, through, and out of a landfill given design parameters and general soil and local climate information.

II. PROCEDURE

The general setup and inputs to the HELP model were kept consistent with a separate analysis performed on this project which used the HELP model to estimate leachate generation in the landfill (see HELP Model Leachate Generation Estimates). The "closed conditions" scenario from that analysis, which modeled the complete landfill from top to bottom, was referenced herein. For this analysis, however, design inputs for only the cover system components were entered. With the exception of the two different cover systems modeled, all other inputs into the model were held constant in order to isolate the correlation of leakage through the cover system directly to the design input parameters for the cover system components. A description of the two design cases modeled in this analysis follows:

The compacted clay cover system is shown in Figure 1, and consists of the following layers, from top to bottom:

- A 12 inch thick vegetative cover soil capable of sustaining native vegetation; and,
- A 24 inch thick compacted clay layer with permeability no greater than 10^{-7} cm/sec.

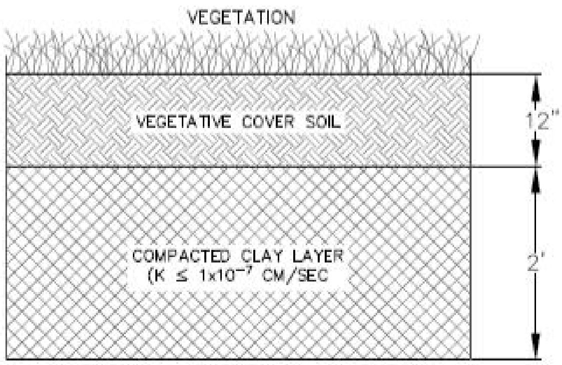


FIGURE 1: COMPACTED CLAY FINAL COVER SYSTEM

NOT TO SCALE

The geosynthetic cover system is shown in Figure 2, and consists of the following layers, from top to bottom:

- A 12 inch thick vegetative cover soil capable of sustaining native vegetation;
- A 12 inch thick protective cover soil layer;
- A double sided geocomposite drainage layer; and,
- A 40 mil linear low density polyethylene (LLDPE) flexible membrane liner barrier layer which has permeability much less than the required minimum 10^{-7} cm/sec.

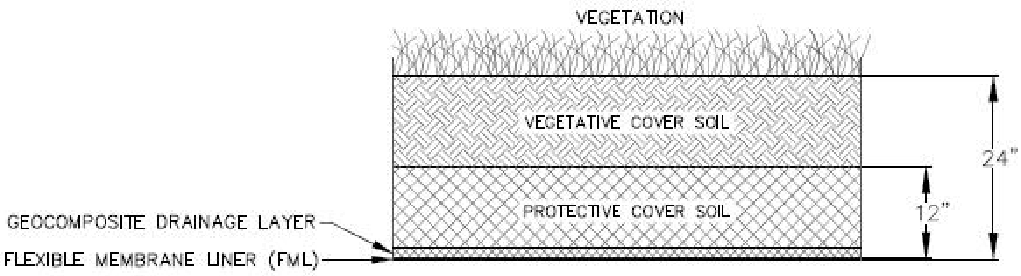


FIGURE 2: GEOSYNTHETIC FINAL COVER SYSTEM

In addition, the following two layers were used in the model for both cover systems mentioned above to represent the actual scenario appropriately:

- A 12 inch thick intermediate cover
- A 165 feet thick waste

Each case was simulated by the HELP Model for a one-hundred (100) year time period to verify that all reasonably expected climactic conditions were modeled. The model presents peak daily results and average annual results over the 100-year simulation period.

III. ASSUMPTIONS

The following assumptions were made in performing the HELP Model analysis. For reference, relevant pages of the HELP Model Engineering Documentation Manual are provided as Attachment A.

Landfill Cover System Material Textures

- Both the vegetative and protective cover layer were modeled as HELP default texture 12 (USCS Classification CL) with a default permeability of $4.2x10^{-5}$ cm/sec.

- The compacted clay layer was modeled as HELP default texture 16 (Liner Soil) with a default permeability of $1.0x10^{-7}$ cm/sec.

- The 200 mil geocomposite drainage layer was modeled as HELP default texture 20 (Lateral Drainage Layer) with a default permeability of 10.00 cm/sec.

- The 40 mil Linear Low Density Polyethylene (LLDPE) geomembrane was modeled as HELP default texture 36 (Flexible membrane liner) with a default permeability of 4.0 x 10^{-13} cm/sec.

- The 12 inch thick intermediate cover was modeled as HELP default texture 12 (Vertical Percolation Layer) with a default permeability of 4.2 x 10^{-5} cm/sec.

- The 165 feet thick Waste layer was modeled as HELP default texture 33 (Vertical Percolation Layer) with a re-defined permeability of 1.0 x 10^{-3} cm/sec.

General Assumptions

- The geosynthetics materials will be constructed with good quality workmanship and in accordance with the project CQA Plan.

- The initial water contents of all layers were manually set equal to the default HELP specified field capacity of the material, which represents the water content of the material after a prolonged period of gravity drainage. However, it should be noted that for the purpose of calculating hydraulic flow through the landfill system, the HELP

Model automatically assumes that all barrier layers (the compacted soil liner and final cover barrier layer) are saturated.

- The HELP Model was utilized to synthetically generate temperature, precipitation, evapotranspiration, and solar radiation data based on a location of Scepter, Indiana. The evaporative zone depth was set to 12 inches because that is the depth of the vegetative cover soil layer.

- The HELP Model results are independent of the landfill area. A one (1) acre area was considered for the analysis. Therefore, cover system leakage results are presented as cubic feet per acre per time period (annual or daily). Results were converted to gallons per acre per time period using the conversion factor listed below:

$$\frac{ft^3}{time\ period} \times \frac{7.48\ gallons}{ft^3} \times \frac{time\ period}{\#\ of\ days}$$

IV. RESULTS

The HELP Model output files for the prescribed and proposed cover systems are provided in Attachments.

The key results of the HELP Model comparing the hydraulic performance of the two cover systems are shown in Table 1 below. Note that these results are presented for comparative purposes and may not represent accurate estimates of actual leakage through the constructed cover system.

Table 1. Hydraulic Performance of Compacted Clay vs. Geosynthetic Final Cover Systems

Final Cover System Case	Average Daily Leakage Rate (gal/acre/day)	Peak Daily Leakage Rate (gal/acre/day)	Average Annual Leakage Rate (gal/acre/year)
Compacted Clay	123.3	138.53	45,006
Geosynthetic	0.04	0.45	13

V. CONCLUSIONS

The results in Table 1 provide a side by side comparison of the compacted clay vs. Geosynthetic final cover systems, which shows that the Geosynthetic final cover system will result in less moisture leakage into the landfill than the compacted clay system. Also, the Geosynthetic final cover system is expected to cost less with easier, more controllable construction methods.

VI. REFERENCES

Koerner, R.M., “Designing with Geosynthetics”, 6th Edition, Xlibris, 2012.

Schroeder, P.R., Lloyd, C.M., and Zappi, P.A., “The Hydraulic Evaluation of Landfill Performance (HELP) Model, User’s Guide for Version 3”, U.S. Environmental Protection Agency, Office of Research and Development, Washington, D.C., Report No. EPA/600/R094/168a, 1994 a.

Schroeder, P.R., Dozier, T.S., Zappi, P.A., McEnroe, B.M., Sjostrom, J.W., and Peyton, R.L., “The Hydraulic Evaluation of Landfill Performance (HELP) Model, Engineering Documentation for Version 3”, U.S. Environmental Protection Agency, Office of Research and Development, Washington, D.C., Report No. EPA/600/R094/168b, 1994 b.

TDEC, “Rules of Tennessee Department of Environment and Conservation, Chapter 0400-11-01 – Solid Waste Processing and Disposal”, Division of Solid Waste Management, November 2008.

ATTACHMENT A

Select HELP Model Engineering Documentation

ATTACHMENT B

HELP Model Results: Compacted Clay Final Cover System

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******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**                HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE            **
**                   HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)            **
**                     DEVELOPED BY ENVIRONMENTAL LABORATORY               **
**                       USAE WATERWAYS EXPERIMENT STATION                 **
**               FOR USEPA RISK REDUCTION ENGINEERING LABORATORY           **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


 PRECIPITATION DATA FILE:     c:\temp\nick\SPT_PREC.D4
 TEMPERATURE DATA FILE:       c:\temp\nick\SPT_TEMC.D7
 SOLAR RADIATION DATA FILE:   c:\temp\nick\SPT_SOLC.D13
 EVAPOTRANSPIRATION DATA:     c:\temp\nick\SPT-EVPC.D11
 SOIL AND DESIGN DATA FILE:   c:\temp\nick\SPT-CAPC.D10
 OUTPUT DATA FILE:            c:\temp\nick\SPT-CAPC.OUT


 TIME:   8: 4     DATE:   6/30/2016


******************************************************************************

      TITLE:  Scepter Waverly Exp 4
```

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*****


      NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
             COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                                LAYER  1
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                     MATERIAL TEXTURE NUMBER  12
      THICKNESS                    =     12.00   INCHES
      POROSITY                     =      0.4710 VOL/VOL
      FIELD CAPACITY               =      0.3420 VOL/VOL
      WILTING POINT                =      0.2100 VOL/VOL
      INITIAL SOIL WATER CONTENT   =      0.4664 VOL/VOL
      EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC
   NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
           FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                                LAYER  2
                                --------

                     TYPE 3 - BARRIER SOIL LINER
                     MATERIAL TEXTURE NUMBER  16
      THICKNESS                    =     24.00   INCHES
      POROSITY                     =      0.4270 VOL/VOL
      FIELD CAPACITY               =      0.4180 VOL/VOL
      WILTING POINT                =      0.3670 VOL/VOL
      INITIAL SOIL WATER CONTENT   =      0.4270 VOL/VOL
      EFFECTIVE SAT. HYD. COND.    =  0.100000001000E-06 CM/SEC


                                LAYER  3
                                --------

                  TYPE 1 - VERTICAL PERCOLATION LAYER
                     MATERIAL TEXTURE NUMBER  12
      THICKNESS                    =     12.00   INCHES
      POROSITY                     =      0.4710 VOL/VOL
      FIELD CAPACITY               =      0.3420 VOL/VOL
      WILTING POINT                =      0.2100 VOL/VOL
      INITIAL SOIL WATER CONTENT   =      0.3420 VOL/VOL
      EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC
```

LAYER 4

TYPE 1 - VERTICAL PERCOLATION LAYER
MATERIAL TEXTURE NUMBER 0

THICKNESS	=	1980.00	INCHES
POROSITY	=	0.3750	VOL/VOL
FIELD CAPACITY	=	0.0550	VOL/VOL
WILTING POINT	=	0.0200	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0558	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000005000E-02	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE #12 WITH A FAIR STAND OF GRASS, A SURFACE SLOPE OF 33.% AND A SLOPE LENGTH OF 100. FEET.

SCS RUNOFF CURVE NUMBER	=	89.20	
FRACTION OF AREA ALLOWING RUNOFF	=	100.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	1.000	ACRES
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.771	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	2.826	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.260	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	130.437	INCHES
TOTAL INITIAL WATER	=	130.437	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
NASHVILLE TENNESSEE

STATION LATITUDE	=	36.12	DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00	
START OF GROWING SEASON (JULIAN DATE)	=	86	
END OF GROWING SEASON (JULIAN DATE)	=	308	
EVAPORATIVE ZONE DEPTH	=	6.0	INCHES

AVERAGE ANNUAL WIND SPEED = 8.00 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY = 68.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY = 69.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY = 75.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY = 71.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR NASHVILLE TENNESSEE

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.49	4.03	5.58	4.47	4.56	3.70
3.82	3.40	3.71	2.58	3.52	4.63

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR NASHVILLE TENNESSEE

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
37.10	40.40	49.00	59.60	68.10	75.80
79.40	78.40	72.30	60.20	48.60	40.90

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR NASHVILLE TENNESSEE AND STATION LATITUDE = 36.12 DEGREES

****

AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 100

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						

TOTALS	4.31	4.65	5.61	4.43	4.53	3.61
	3.89	3.49	3.94	2.43	3.57	4.50
STD. DEVIATIONS	1.84	2.17	2.23	1.81	2.12	1.63
	2.05	2.00	2.35	1.59	1.68	2.25

RUNOFF

TOTALS	2.890	3.168	2.842	1.106	0.949	0.339
	0.501	0.484	1.098	0.613	1.501	2.910
STD. DEVIATIONS	1.899	2.148	2.285	1.147	1.069	0.515
	0.935	0.859	1.523	0.997	1.486	2.205

EVAPOTRANSPIRATION

TOTALS	1.258	1.557	3.001	3.589	3.566	3.388
	3.182	2.879	2.371	1.566	1.367	1.191
STD. DEVIATIONS	0.273	0.414	0.442	0.866	1.349	1.311
	1.347	1.221	0.907	0.622	0.265	0.197

PERCOLATION/LEAKAGE THROUGH LAYER 2

TOTALS	0.1515	0.1361	0.1467	0.1357	0.1358	0.1285
	0.1314	0.1318	0.1300	0.1377	0.1407	0.1518
STD. DEVIATIONS	0.0044	0.0055	0.0046	0.0051	0.0042	0.0035
	0.0039	0.0042	0.0054	0.0081	0.0077	0.0049

PERCOLATION/LEAKAGE THROUGH LAYER 4

TOTALS	0.0273	0.0236	0.0259	0.0246	0.0260	0.0265
	0.0287	0.0301	0.0298	0.0318	0.0304	0.0308

STD. DEVIATIONS 0.0459 0.0402 0.0443 0.0424 0.0447
0.0460
0.0495 0.0514 0.0502 0.0531 0.0506
0.0510

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 2

AVERAGES 10.4718 9.9921 9.3935 7.9046 6.8981
6.2306
5.9018 5.9935 6.5862 7.3380 9.0819
10.5424

STD. DEVIATIONS 0.9918 1.3313 1.0513 1.2071 0.9556
0.8311
0.8946 0.9663 1.2774 1.8533 1.8217
1.1057

****

****

AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 100

	INCHES	CU. FEET	PERCENT
PRECIPITATION	48.97 (7.932)	177771.3	100.00
RUNOFF	18.401 (5.6970)	66794.05	37.573
EVAPOTRANSPIRATION	28.916 (3.5645)	104963.33	59.044
PERCOLATION/LEAKAGE THROUGH	1.65754 (0.02668)	6016.854	3.38460

LAYER 2

AVERAGE HEAD ON TOP 8.028 (0.517)
OF LAYER 2

PERCOLATION/LEAKAGE THROUGH 0.33545 (0.56815) 1217.675
0.68497
LAYER 4

CHANGE IN WATER STORAGE 1.321 (0.9885) 4796.18
2.698

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******

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*****************************************************************************

          PEAK DAILY VALUES FOR YEARS      1 THROUGH  100
    ------------------------------------------------------------------------
---
                                              (INCHES)      (CU. FT.)
                                             ----------   ------------
-
       PRECIPITATION                            5.65          20509.500

       RUNOFF                                   5.643         20484.9766

       PERCOLATION/LEAKAGE THROUGH LAYER  2     0.005102
18.52118

       AVERAGE HEAD ON TOP OF LAYER  2        12.000

       PERCOLATION/LEAKAGE THROUGH LAYER  4     0.005269
19.12794

       SNOW WATER                               7.79          28269.1699

       MAXIMUM VEG. SOIL WATER (VOL/VOL)                0.4710

       MINIMUM VEG. SOIL WATER (VOL/VOL)                0.2100

*****************************************************************************
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****

FINAL WATER STORAGE AT END OF YEAR 100

LAYER	(INCHES)	(VOL/VOL)
1	5.5140	0.4595
2	10.2480	0.4270
3	4.1040	0.3420
4	242.6973	0.1226
SNOW WATER	0.000	

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****

ATTACHMENT C

HELP Model Results: Geosynthetic Final Cover System

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**
 **              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE
**
 **                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)
**
 **                  DEVELOPED BY ENVIRONMENTAL LABORATORY
**
 **                    USAE WATERWAYS EXPERIMENT STATION
**
 **               FOR USEPA RISK REDUCTION ENGINEERING LABORATORY
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 PRECIPITATION DATA FILE:    c:\temp\nick\SPT_PREC.D4
 TEMPERATURE DATA FILE:      c:\temp\nick\SPT_TEMC.D7
 SOLAR RADIATION DATA FILE:  c:\temp\nick\SPT_SOLC.D13
 EVAPOTRANSPIRATION DATA:    c:\temp\nick\SPT-EVPC.D11
 SOIL AND DESIGN DATA FILE:  c:\temp\nick\SPT-CAPG.D10
 OUTPUT DATA FILE:           c:\temp\nick\SPT-CAPG.OUT


 TIME:   8: 5     DATE:   6/30/2016



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      TITLE:  Scepter Waverly Exp 4
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NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER WERE
         COMPUTED AS NEARLY STEADY-STATE VALUES BY THE PROGRAM.


                            LAYER  1
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              TYPE 1 - VERTICAL PERCOLATION LAYER
                  MATERIAL TEXTURE NUMBER  12
  THICKNESS                    =     24.00   INCHES
  POROSITY                     =      0.4710 VOL/VOL
  FIELD CAPACITY               =      0.3420 VOL/VOL
  WILTING POINT                =      0.2100 VOL/VOL
  INITIAL SOIL WATER CONTENT   =      0.4131 VOL/VOL
  EFFECTIVE SAT. HYD. COND.    =  0.419999997000E-04 CM/SEC
NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  3.00
         FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.


                            LAYER  2
                            --------

                TYPE 2 - LATERAL DRAINAGE LAYER
                  MATERIAL TEXTURE NUMBER  20
  THICKNESS                    =      0.20   INCHES
  POROSITY                     =      0.8500 VOL/VOL
  FIELD CAPACITY               =      0.0100 VOL/VOL
  WILTING POINT                =      0.0050 VOL/VOL
  INITIAL SOIL WATER CONTENT   =      0.0100 VOL/VOL
  EFFECTIVE SAT. HYD. COND.    =   10.0000000000     CM/SEC
  SLOPE                        =     33.30   PERCENT
  DRAINAGE LENGTH              =    100.0    FEET


                            LAYER  3
                            --------

               TYPE 4 - FLEXIBLE MEMBRANE LINER
                  MATERIAL TEXTURE NUMBER  36
  THICKNESS                    =      0.04   INCHES
  POROSITY                     =      0.0000 VOL/VOL
  FIELD CAPACITY               =      0.0000 VOL/VOL
  WILTING POINT                =      0.0000 VOL/VOL
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INITIAL SOIL WATER CONTENT  =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.399999993000E-12 CM/SEC
FML PINHOLE DENSITY         =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =      3.00   HOLES/ACRE
FML PLACEMENT QUALITY       =  3 - GOOD


                      LAYER  4
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           TYPE 1 - VERTICAL PERCOLATION LAYER
              MATERIAL TEXTURE NUMBER   0
THICKNESS                   =   1980.00   INCHES
POROSITY                    =      0.3750 VOL/VOL
FIELD CAPACITY              =      0.0550 VOL/VOL
WILTING POINT               =      0.0200 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0550 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.100000005000E-02 CM/SEC


          GENERAL DESIGN AND EVAPORATIVE ZONE DATA
          ----------------------------------------

  NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT
           SOIL DATA BASE USING SOIL TEXTURE #12 WITH A
           FAIR STAND OF GRASS, A SURFACE SLOPE OF 33.%
           AND A SLOPE LENGTH OF  100. FEET.

SCS RUNOFF CURVE NUMBER              =     89.20
FRACTION OF AREA ALLOWING RUNOFF     =    100.0    PERCENT
AREA PROJECTED ON HORIZONTAL PLANE   =      1.000  ACRES
EVAPORATIVE ZONE DEPTH               =      6.0    INCHES
INITIAL WATER IN EVAPORATIVE ZONE    =      2.198  INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE   =      2.826  INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE   =      1.260  INCHES
INITIAL SNOW WATER                   =      0.000  INCHES
INITIAL WATER IN LAYER MATERIALS     =    118.816  INCHES
TOTAL INITIAL WATER                  =    118.816  INCHES
TOTAL SUBSURFACE INFLOW              =      0.00   INCHES/YEAR


             EVAPOTRANSPIRATION AND WEATHER DATA
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  NOTE:  EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
            NASHVILLE             TENNESSEE
```

STATION LATITUDE	=	36.12 DEGREES
MAXIMUM LEAF AREA INDEX	=	2.00
START OF GROWING SEASON (JULIAN DATE)	=	86
END OF GROWING SEASON (JULIAN DATE)	=	308
EVAPORATIVE ZONE DEPTH	=	6.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	8.00 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	68.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	69.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	75.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	71.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NASHVILLE TENNESSEE

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
4.49	4.03	5.58	4.47	4.56	3.70
3.82	3.40	3.71	2.58	3.52	4.63

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NASHVILLE TENNESSEE

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
37.10	40.40	49.00	59.60	68.10	75.80
79.40	78.40	72.30	60.20	48.60	40.90

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR NASHVILLE TENNESSEE
AND STATION LATITUDE = 36.12 DEGREES

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 100

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	4.31	4.65	5.61	4.43	4.53	3.61
	3.89	3.49	3.94	2.43	3.57	4.50
STD. DEVIATIONS	1.84	2.17	2.23	1.81	2.12	1.63
	2.05	2.00	2.35	1.59	1.68	2.25
RUNOFF						
TOTALS	0.761	1.126	1.133	0.478	0.596	0.260
	0.413	0.372	0.671	0.230	0.387	0.748
STD. DEVIATIONS	0.802	1.337	1.405	0.484	0.691	0.323
	0.638	0.513	0.834	0.386	0.469	0.887
EVAPOTRANSPIRATION						
TOTALS	1.249	1.511	2.654	3.043	3.044	3.043
	2.934	2.598	2.125	1.451	1.343	1.191
STD. DEVIATIONS	0.281	0.414	0.528	0.832	1.151	1.144
	1.170	1.039	0.793	0.616	0.291	0.197
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	2.3535	2.1502	2.1723	1.3011	0.9589	0.6044
	0.4741	0.5071	0.8558	0.7834	1.2500	2.2100
STD. DEVIATIONS	1.3226	1.4209	1.1949	0.7984	0.6857	0.4313
	0.5231	0.5337	0.7732	0.7862	1.0238	1.4211

PERCOLATION/LEAKAGE THROUGH LAYER 3

```
    TOTALS                  0.0001   0.0001   0.0001   0.0000   0.0000
0.0000
                            0.0000   0.0000   0.0000   0.0000   0.0000
0.0001

    STD. DEVIATIONS         0.0000   0.0000   0.0000   0.0000   0.0000
0.0000
                            0.0000   0.0000   0.0000   0.0000   0.0000
0.0000

  PERCOLATION/LEAKAGE THROUGH LAYER  4
  -----------------------------------
    TOTALS                  0.0001   0.0001   0.0001   0.0000   0.0000
0.0000
                            0.0000   0.0000   0.0000   0.0000   0.0000
0.0001

    STD. DEVIATIONS         0.0000   0.0000   0.0000   0.0000   0.0000
0.0000
                            0.0000   0.0000   0.0000   0.0000   0.0000
0.0000


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               AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)
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  DAILY AVERAGE HEAD ON TOP OF LAYER  3
  -------------------------------------
    AVERAGES                0.0013   0.0013   0.0012   0.0007   0.0005
0.0003
                            0.0003   0.0003   0.0005   0.0004   0.0007
0.0013

    STD. DEVIATIONS         0.0008   0.0009   0.0007   0.0005   0.0004
0.0002
                            0.0003   0.0003   0.0005   0.0004   0.0006
0.0009


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     AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS    1 THROUGH
100
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                                          INCHES            CU. FEET
PERCENT
                                 -------------------   -------------    ---
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  PRECIPITATION                   48.97     (   7.932)    177771.3
100.00

  RUNOFF                           7.174    (  3.0214)     26042.17
14.649

  EVAPOTRANSPIRATION              26.185    (  3.1329)     95052.89
53.469

  LATERAL DRAINAGE COLLECTED      15.62071 (  3.92088)     56703.164
31.89669
    FROM LAYER  2

  PERCOLATION/LEAKAGE THROUGH      0.00050 (  0.00010)         1.803
0.00101
    LAYER  3

  AVERAGE HEAD ON TOP              0.001 (    0.000)
    OF LAYER  3

  PERCOLATION/LEAKAGE THROUGH      0.00050 (  0.00010)         1.805
0.00102
    LAYER  4

  CHANGE IN WATER STORAGE         -0.008    (  1.2461)       -28.77      -
0.016

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                PEAK DAILY VALUES FOR YEARS    1 THROUGH  100
    ------------------------------------------------------------------------
---
                                                   (INCHES)      (CU. FT.)
                                                  ----------   ------------
-
       PRECIPITATION                                5.65         20509.500

       RUNOFF                                       4.717        17122.2246

       DRAINAGE COLLECTED FROM LAYER  2             0.91963
3338.24487

       PERCOLATION/LEAKAGE THROUGH LAYER  3         0.000016
0.05951

       AVERAGE HEAD ON TOP OF LAYER  3              0.020

       MAXIMUM HEAD ON TOP OF LAYER  3              0.030

       LOCATION OF MAXIMUM HEAD IN LAYER  2
             (DISTANCE FROM DRAIN)                  0.0 FEET

       PERCOLATION/LEAKAGE THROUGH LAYER  4         0.000007
0.02623

       SNOW WATER                                   7.79         28269.1699


       MAXIMUM VEG. SOIL WATER (VOL/VOL)                    0.4680

       MINIMUM VEG. SOIL WATER (VOL/VOL)                    0.2100

        ***  Maximum heads are computed using McEnroe's equations.  ***

             Reference:  Maximum Saturated Depth over Landfill Liner
                         by Bruce M. McEnroe, University of Kansas
                         ASCE Journal of Environmental Engineering
                         Vol. 119, No. 2, March 1993, pp. 262-270.


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FINAL WATER STORAGE AT END OF YEAR 100

LAYER	(INCHES)	(VOL/VOL)
1	9.1216	0.3801
2	0.0020	0.0100
3	0.0000	0.0000
4	108.9000	0.0550
SNOW WATER	0.000	

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APPENDIX F.3B

Final Cover System Geocomposite Design

AECOM

Job	Scepter Waverly Landfill East Phase Site	Project No.	60398526 20500022. 00002	Sheet	1 of 3
Description	Final Cover System	Computed by	MSM	Date	06/24/16
	Geocomposite Design	Checked by	NSG	Date	07/01/16

I. PURPOSE

The purpose of this analysis is to calculate the required transmissivity of the final cover system geocomposite. The following sections summarize the methodology, assumptions, and results of the final cover system geocomposite design for the proposed Landfill facility. For further detail on the specific calculations performed, refer to the corresponding input/output data provided in the attachments.

II. METHODOLOGY

Slope geometry and cover system materials were established from the proposed permit design drawings and are summarized below.

The cap system will be installed at the final surface of the crest and sideslopes overlying the waste material. The proposed maximum sideslope angle for the proposed cap system is 3 Horizontal to 1 Vertical (3H:1V). The longest 3H:1V slope length is approximately 90 feet, with storm water terraces spaced on the 3H:1V slope every 30 vertical feet maximum. Water collected in the Geocomposite will be drained into the storm water terraces.

Layers and layer thicknesses for the final cover system cap are anticipated as follows:

Table 1. Layer Summary for the Final Cover System

THICKNESS	LAYER
Final Cover	
6 in	Vegetative Cover Soil
18 in	Protective Cover Soil
n/a	Geocomposite Drainage Layer
n/a	Textured Geomembrane

n/a – thickness of layer is small (negligible)

The geocomposite must be designed to transmit the expected flow of water into the geocomposite through the overlying cover soil. It is assumed that the maximum flow into the geocomposite will occur when the overlying soil is saturated. The cover soil was modeled with a conservative long-term permeability (k_c) of $5x10^{-5}$ cm/sec. The following equation can be used to model the relationship between the average head level in the geocomposite (h_{avg}), the slope length (L) and angle (β), the permeability of the cover soil (k_c) and the required permeability of the geocomposite (k_d).:

$$h_{avg} = \frac{k_c L(\cos\beta)}{k_d(\sin\beta)} \qquad (1)$$

The minimum required transmissivity (T_{design}) of the geocomposite drainage layer is determined by limiting the average head (h_{avg}) on the drainage layer to the thickness of the drainage layer (t_d). For the purpose of calculations, a geocomposite thickness of 0.5 cm (0.2 in or 200 mils) was utilized. Limiting the average head to the approximate thickness of the drainage layer ensures drainage occurs within the drainage layer.

The minimum required transmissivity (T_{design}) of the geocomposite is calculated using the following equation (from "Designing with GRI Standard GC8," Narejo and Richardson, 2003):

$$T_{design} = k_d t_d \qquad (2)$$

Where,

t_d = Thickness of the Drainage Layer

A factor of safety is then applied to T_{design} to obtain the allowable transmissivity (T_{allow}) (from "Designing with GRI Standard GC8," Narejo and Richardson, 2003), as shown in the equation below:

$$FS = \frac{T_{allow}}{T_{design}} \qquad (3)$$

Reduction factors are then applied to the allowable transmissivity (T_{allow}), which represents long-term in-situ conditions. The decrease in flow capacity from the minimum required transmissivity (T_{spec}) to the long-term in-situ conditions is described by reduction factors (RF) as given in "GSI White Paper #4: Reduction Factors Used in Geosynthetic Design" (Koerner and Koerner, 2005). The equation below was used to determine T_{spec}:

$$T_{allow} = \frac{T_{spec}}{RF_{IN} RF_{CR} RF_{CC} RF_{BC}} \qquad (4a)$$

Substituting equation (3) and solving for T_{spec}:

$$T_{spec} = T_{design} FS \times RF_{IN} RF_{CR} RF_{CC} RF_{BC} \qquad (4b)$$

Typical values for reduction factors for landfill covers from Koerner and Koerner (2005) and Narejo and Richardson (2003) are included in Attachment A. Values chosen for reduction factors were taken from the range of values presented and are summarized below:

Table 2. Reduction Factor Summary

Reduction Factor	Value	Comments
Intrusion, RF_{IN}	1.0	Boundary conditions during laboratory testing will account for intrusion; 1.0 recommended by Narejo (2004) – included in Attachment A
Creep, RF_{CR}	1.1	Low loading conditions
Chemical Clogging, RF_{CC}	1.2	Potential for some precipitate from onsite cover soil (clay from weathered chert)
Biological Clogging, RF_{BC}	2.3	Middle of range from Narejo (2004)
Drainage Factor of Safety, FS	3	Conventionally between 2 and 3

III. RESULTS OF ANALYSIS

As discussed in Section II, the geocomposite drainage layer must be selected with adequate transmissivity to limit the depth of flow to the thickness of the geocomposite. Conservative assumptions regarding factors of safety, reduction factors, and the assumed saturated hydraulic conductivity of the overlying soils are considered when calculating the specified minimum transmissivity of the final cover system geocomposite.

The design drainage length is approximately 90 feet (length of longest slope) between drainage daylights at each storm water terrace. The minimum transmissivity required to maintain drainage inside the geocomposite on the 3H:1V slopes is $\underline{3.75 \times 10^{-4}\ m^2/sec}$. This is the minimum required value for testing and manufacturer's specifications, (T_{spec}). Refer to Attachment B for supporting calculations.

IV. REFERENCES

Narejo, D. and Richardson, G. (2003), "Designing with GRI Standard GC8." GFR, vol. 21, no. 6, pp. 20–23.

Koerner, R. M. and Koerner, G. R. (2005), "GSI White Paper #4 Reduction Factors (RFs) Used in Geosynthetic Design." Geosynthetic Institute

ATTACHMENT A

Application Area	Range of Reduction Factor Values			
	RF_{IN}	RF_{CR}*	RF_{CC}	RF_{BC}
Sport fields	1.0 to 1.2	1.0 to 1.5	1.0 to 1.2	1.1 to 1.3
Capillary breaks	1.1 to 1.3	1.0 to 1.2	1.1 to 1.5	1.1 to 1.3
Roof and plaza decks	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2	1.1 to 1.3
Retaining walls, seeping rock, and soil slopes	1.3 to 1.5	1.2 to 1.4	1.1 to 1.5	1.0 to 1.5
Drainage blankets	1.3 to 1.5	1.2 to 1.4	1.0 to 1.2	1.0 to 1.2
Infiltrating water drainage for landfill covers	1.3 to 1.5	1.1 to 1.4	1.0 to 1.2	1.5 to 2.0
Secondary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0
Primary leachate collection (landfills)	1.5 to 2.0	1.4 to 2.0	1.5 to 2.0	1.5 to 2.0
Wick Drains (PVDs)	1.5 to 2.5	1.0 to 2.5	1.0 to 1.2	1.0 to 1.2
Highway edge drains	1.2 to 1.8	1.5 to 3.0	1.1 to 5.0	1.0 to 1.2

From Koerner R., and Koerner G. (2005) "GSI White Paper #4: Reduction Factors Used in Geosynthetic Design." Geosynthetic Institute.

LONG-TERM PERFORMANCE CONSIDERATIONS FOR GEONET DRAINAGE GEOCOMPOSITES

Dhani Narejo, GSE Lining Technology Inc., Houston, USA

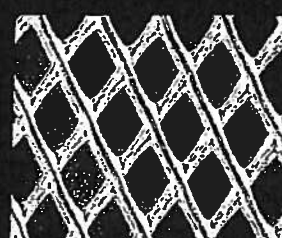
ABSTRACT
Drainage geocomposites have gained increasing acceptance within the engineering community as the material of choice for the lateral conveyance of liquids and gases. The hydraulic performance of these materials is typically expressed as transmissivity or flow rate at site-specific gradient, normal stress and boundary conditions. However, since these materials are visco-elastic in nature, compressive creep can significantly affect their long-term hydraulic performance. In addition to creep, there is the potential for the chemical and biological clogging of the filter geotextile and the geonet drainage core. Over the last several years, significant progress has been made in characterizing the engineering properties of geonet drainage geocomposites and developing models to predict their long-term behaviour on the basis of short-term laboratory tests. Additional work is needed in the area of chemical and biological clogging to further supplement the current information. In addition, the impact of leachate recirculation and higher temperatures in bioreactor landfills on the long-term performance of geocomposites merits further study.

RÉSUMÉ
Drainage geocomposites have gained increasing acceptance within the engineering community as the material of choice for the lateral conveyance of liquids and gases. The hydraulic performance of these materials is typically expressed as transmissivity or flow rate at site-specific gradient, normal stress and boundary conditions. However, since these materials are visco-elastic in nature, compressive creep can significantly affect their long-term hydraulic performance. In addition to creep, there is the potential for the chemical and biological clogging of the filter geotextile and the geonet drainage core. Over the last several years, significant progress has been made in characterizing the engineering properties of geonet drainage geocomposites and developing models to predict their long-term behaviour on the basis of short-term laboratory tests. Additional work is needed in the area of chemical and biological clogging to further supplement the current information. In addition, the impact of leachate recirculation and higher temperatures in bioreactor landfills on the long-term performance of geocomposites merits further study.

1. INTRODUCTION

A geonet drainage geocomposite consists of a geonet core and a geotextile, where the geotextile is heat-laminated to one or both sides of the geonet. The geonet is made of extruded High Density Polyethylene (HDPE) in a manner that forms a relatively open structure ideal for the in-plane transmission of liquids and/or gases. The geotextile serves as a filter and separator, while the geonet core is intended to provide the lateral flow capacity. Geotextiles currently used for this purpose are almost exclusively of the nonwoven needlepunched type made from polypropylene (PP) or polyester (PE) resins. Geonet drainage geocomposites are differentiated primarily by the structure of the geonet as illustrated in Figures 1 (a) and (b).

Drainage geocomposites are used predominantly in environmental applications such as landfills and lagoons. However, there is growing interest in the use of these materials in such civil engineering applications as roadways, buildings, canals, etc. Landfills – the dominant market segment for these materials – are characterized by relatively large areas with slopes ranging from as low as 2% to as high as 33%. Specifically, there are four applications in landfills where drainage geocomposites are utilized: i) landfill cover drainage layer, ii) landfill gas vent layer, iii) landfill leachate collection and removal



(a) biplanar geonet

(a) triplanar geonet

Figure 1 Plan view of biplanar and triplanar geonets.

layer, and iv) landfill leakage detection layer. The design of each of these layer may involve the following performance properties of the drainage geocomposite: i) flow rate or transmissivity (heretofore referred to as transmissivity), ii) interface shear strength, and iii) filtration properties (including "filtration opening size" and permeability). This paper deals with only one of the above three performance characteristics, namely transmissivity.

The transmissivity of drainage geocomposites is a function of available pore-space as illustrated in Figure 2. Any mechanism that tends to reduce this pore space would decrease geocomposite transmissivity. Currently known factors include the following: i) geonet creep, ii) geotextile intrusion into the core structure, iii) chemical clogging within the core, and iv) biological clogging within the core. The reader should note that the concern with biological and chemical clogging of the drainage geocomposite core is differentiated here from a similar concern for the drainage geocomposite filter geotextile. Although mechanisms involved may be similar, the testing and design must be performed separately for the filter and drainage media.

Figure 2 Cross-section of a biplanar drainage geocomposite.

2. TRANSMISSIVITY AND REDUCTION FACTORS

Transmissivity is defined as the flow rate of water transmitted through a unit width of the product under a specific hydraulic gradient as measured in a laboratory test. The transmissivity test is performed using the type of equipment shown schematically in Figure 3. For the test to provide a transmissivity value that can be used in design, the specimen top and bottom boundaries as well as the gradient should be the same as in the field. The test is typically continued for a reasonably long enough time to include the effect of initial compression, and intrusion of geotextile into the geonet structure. The current state-of-the-practice in the US is represented by GRI GC8 which requires the test to be continued for 100 hours. The resulting value is then modified to include the effect of creep, chemical clogging and biological clogging as in Equation 1 (from GRI GC8, 2001):

$$\theta_{allow} = \frac{\theta_{100}}{RF_{cr} x RF_{cc} x RF_{bc}} \qquad [1]$$

where θ_{allow} = allowable transmissivity for the specific product being considered (m^2/sec), θ_{100} = 100-hour performance transmissivity from actual test, RF_{cr} = reduction factor for creep of the geonet core, RF_{cc} = reduction factor for chemical clogging, RF_{bc} = reduction factor for biological clogging.

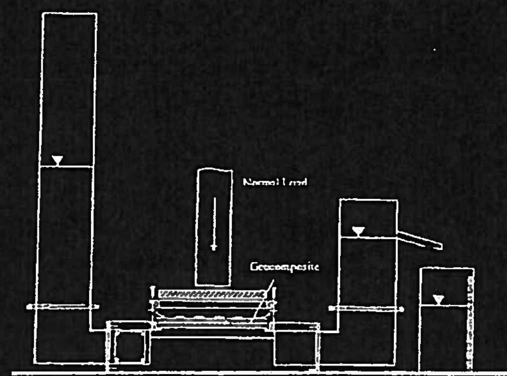


Figure 3 Schematics of the transmissivity test (Richardson et. al., 2000).

It must be noted here that certain versions of Equation 1 use such additional reduction factors as geotextile intrusion into the geonet structure and for "particulate clogging" of the geonet core. It is the author's opinion that a reduction factor for intrusion may not be necessary as the performance transmissivity test already includes this effect. The concern regarding particulate clogging of drainage core can, and should, be addressed by proper geotextile filter design so that fines do not pass the geotextile in the first place. This should then be supplemented with proper construction quality assurance (CQA) procedures that minimize infiltration of dust into the drainage core during the installation process.

The allowable value of transmissivity from Equation 1 must then be compared with "required transmissivity" to calculate a factor of safety as provided in the equation below:

$$FS = \frac{\theta_{allow}}{\theta_{req}} \qquad [2]$$

where FS = factor of safety for drainage, and θ_{req} = required transmissivity (m^2/sec) for a specific project.

The three reduction factors in the denominator of Equation 1 along with the performance transmissivity value (θ_{100}) determine whether a particular product is acceptable for a given project. It is recognized that this decision can be only as good as the quality of the data used to arrive at the reduction factors. The state-of-the-practice, limitations of current approach and the need for future research on reduction factors is discussed in the following sections.

2.1 Reduction Factor for Creep, RF_{cr}

Reduction factor for creep is intended to account for the time-dependent compression of the geonet core component of the geocomposite. It should be based on actual testing of the geonet core component of the geocomposite. Geonets can be tested for creep according to one of the two methods currently being used in the industry: a) conventional method, and b) accelerated method. The main difference between the two procedures is the test temperature. In conventional creep method, tests are performed at ambient temperature of around 20 degrees Celsius or any other site-specific temperature. In the accelerated procedure, the testing is performed at several elevated temperatures and the resulting data is then extrapolated to the ambient temperature through time-temperature superposition. Further details of creep testing and the associated calculations can be found in Narejo & Allen (2004). The advantage of the accelerated testing over conventional methods is that the required information can be obtained within hours versus the 14 months required by the conventional tests. Moreover, accelerated testing means that different product formulations and variations can be evaluated economically and within a reasonable time, and more data can be generated for statistical analysis.

Irrespective of whether accelerated or conventional creep testing is performed, the resulting information is of the form presented in Figure 4. For the product and test conditions represented by Figure 4, creep rate is constant at any given normal stress. However, the creep rate increases with an increase in normal stress. Since the creep rate is linear on a semi-log scale, the curves can be extended to obtain thickness at the design life of a project, say 50 years. This value of thickness can then be used to calculate the creep reduction factor, RF_{cr} (Narejo & Allen, 2004) for site-specific stress. Depending on the quality of the product, this creep reduction factor is typically around 1.1 to 1.2 for low stress (<50 kPa) but can be close to 2 for pressures higher than 700 kPa (Narejo & Allen, 2004).

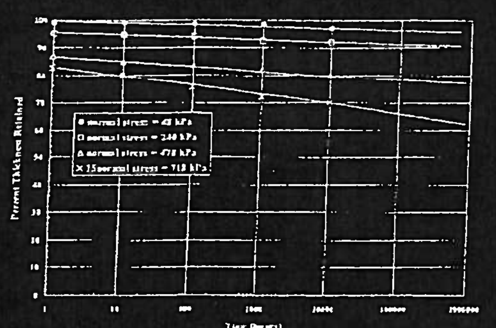


Figure 4 Typical creep response of biplanar geonets.

The creep of geonets is influenced significantly by the physical properties and structure of the geonet, including rib structure, mass, thickness, etc. As such, creep data for geonets is typically specific to products and its generalization is currently not possible.

2.2 Reduction Factor for Chemical Clogging, RF_{cc}

Chemical clogging of drainage materials in landfills results from chemical processes such as the precipitation of calcium carbonate, manganese carbonate and other insoluble substances (e.g., sulfides, chlorides and silicates). A reduction factor for chemical clogging, RF_{cr}, is intended to account for the influence of chemical clogging on the transmissivity of drainage geocomposites. Current industry practice, at least within the US, is to use reduction factors for chemical clogging proposed in the textbook *Designing with Geosynthetics* (Koerner, 1998) and GRI procedure GC8. The values are reproduced in Table 1.

Table 1 Chemical clogging reduction factors (*from* Koerner 1998 and GRI GC8).

Application	Reduction Factor for Chemical Clogging (RF_{cc})
Landfill covers	1.0 to 1.2
Primary leachate collection	1.5 to 2.0
Secondary leachate collection	1.1 to 1.5

2.3 Reduction Factor for Biological Clogging, RF_{bc}

Biological clogging refers to the growth of micro-organisms on and within the drainage media. Biological growth depends on the presence of a suitable bio-chemical environment and nutrients which sustain growth. The biomass growth within the drainage media would reduce the opening size and, hence transmissivity. A reduction factor for biological clogging, RF_{bc}, is used to account for the influence of the biological clogging on geocomposite transmissivity. Currently, the only sources of reference on biological clogging of drainage geocomposites are the geosynthetics textbook – *Designing with Geosynthetics* – and GRI GC8. Suggested reduction factors for biological clogging from these two sources are cited in Table 2.

Table 2 Biological clogging reduction factors (*from* Koerner, 1998, and GRI GC8).

Application	Reduction Factor for Biological Clogging (RF_{bc})
Landfill covers	1.2 to 3.5
Primary leachate collection	1.1 to 1.3
Secondary leachate collection	1.1 to 1.3

3. CRITIQUE OF REDUCTION FACTORS AND RECOMMENDATIONS FOR FURTHER RESEARCH

Reduction factor for creep can be tested in the laboratory with a reasonable degree of confidence as the site conditions can be conveniently modelled. The main variable in creep testing is normal stress, which is determined from the layout and the final contours of the site. As such geosynthetic manufacturers have been testing geonets for creep and a reasonable amount of data already exists. Unfortunately, creep results for geonets are product-specific and each commercially available geonet must be evaluated separately. The SIM Method offers a technique which can help generate a significant amount of data at a reasonable cost. However, manufacturers must demonstrate the validity of this method by performing comparable tests with the conventional technique.

Chemical and biological clogging is very difficult to model in the laboratory. The main reason for this is that the bio-chemical environment for each site may be different. Hence it is difficult to develop a test program the results of which can then be applied uniformly to the design process. It is for this reason that most of the published literature on this topic is of qualitative nature as far as its utilization during the design process is concerned. There is a need for more extensive testing that examines the basic process of clogging in what may be idealized or extreme conditions. This information may then be used to make an "educated guess" about a particular site based on anticipated waste stream and hydrologic conditions.

4. ELEVATED TEMPERATURES AND LEACHATE RECIRCULATION

Bioreactor landfills involve leachate recirculation to accelerate decomposition of the waste mass. Leachate recirculation poses two important challenges to the use of drainage geocomposites: i) elevated temperatures, and ii) higher flow requirements. Elevated temperatures would tend to increase reduction factors for creep, thus lowering the allowable transmissivity. However, the required transmissivity itself may need to be increased beyond that for conventional projects to account for a higher flow of liquid through the drainage layer. Not much is known at this time about the response of drainage geocomposites to leachate circulation. Much research needs to be done in this area to develop recommendations for the design purpose.

5. SUMMARY AND RECOMMENDATIONS

The long-term hydraulic performance of drainage geonets and geocomposites depends on many material as well as site characteristics. A performance transmissivity test provides a 100-hour transmissivity or flow-rate value which can then be further modified to account for site-specific and time-dependent factors. In this regard, there are three specific reductions factors of creep, chemical clogging and biological clogging. Geosynthetic manufacturers have been performing creep tests on their products to develop information on creep reduction factors. However, very limited information is available on biological clogging and chemical clogging of drainage materials. Manufacturers and academics should collaborate to develop further information in this regard. It must be recognized that a model that represents "general" application conditions is very difficult to develop. On the other hand, the tendency to use extremely aggressive conditions in the models provides little practical information for designer. Instead, it may be useful to perform idealized set of testing which can then be analyzed to develop general recommendations for the purpose of design.

6. REFERENCES

GRI GC8. 2001. Geosynthetic Research Institute. Standard guide for determination of the allowable flow rate of a drainage geocomposite.

Koerner, R., 1998. Designing with Geosynthetics. Fourth Edition. Prentice Hall, New Jersey, USA.

Narejo, D., and Allen, S., 2004. Using the Stepped Isothermal Method for geonet creep evaluation. Third European Geosynthetic Conference, 01-03 March 2004, Munich, Germany, pp. 539-544.

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ATTACHMENT B

Final Cover System Geocomposite Design

COMPUTATION OF HEAD ON WEAKEST INTERFACE FOR STATIC UNDRAINED ANALYSIS - 3H:1V SLOPE

Symbol	Description	Values	Units	US units	Comments
h avg	average head	see below	cm		
P	precipitation	0.002350	cm/sec	3.33	*100 yr/1 hr storm- final*
RC	runoff coefficient	0.74		74	**** assumes 75% grass cover, type C soil*
β	angle of slope	18.43	degrees	33.3	*input percent slope in US units cell*
L	slope length	2,743	cm	90	*longest length betwn drainage outlets to be conservative*
k_d	permeability of drainage layer	8.23E-01	cm/sec		*Value required to make $h_{avg} = t_d$*
t_d	thickness of drainage layer	0.50	cm	0	
k_c	permeability of protective material	5.00E-05	cm/sec		*Conservative long term cover permeability*
t_c	thickness of protective material	45.72	cm	1.5	
RF_{IN}	Reduction Factor for intrusion	1.00			*Narejo (2004)*
RF_{CR}	Reduction Factor for creep	1.10			*Low loading conditions*
RF_{CC}	Reduction Factor for chemical clogging	1.20			*Precipitate not anticipated*
RF_{BC}	Reduction Factor for biological clogging	2.30			*Middle of range from Narejo (2004)*
FS	Factor of Safety	3.00			*Conservative, 2 is standard*
T_{design}	Design/Minimum Transmissivity =kd*Tc	4.11E-05	m^2/sec		
T_{spec}	Spec./Lab Transmissivity =ΣRF*Tdesign*FS	3.75E-04	m^2/sec		

Following equations were used:

$$h\ avg = \frac{P(1-RC)\,L(\cos\beta)}{k_d(\sin\beta)}$$ **equation 1a**

if P(1-RC) > kc $$h\ avg = \frac{k_c\,L(\cos\beta)}{k_d(\sin\beta)}$$ **equation 1b**

if h avg from 1b > t_d

then h avg = $t_d + t_c$ **equation 1c**

h avg	6.11	***from equation 1a***
P(1-RC)	0.0006	
kc	0.0001	
h avg	0.5000	***from equation 1b***
h avg	FALSE	***from equation 1c***
h avg	***0.500***	***(cm) used in calculating FS***

APPENDIX F.4

Geotechnical Calculations

F.4A Shallow Translational Slope Stability Analysis (Rev 3)

F.4B Liquefaction Analysis (Rev 3)

F.4C Deep-Seated Slope Stability (Global) Analysis (Rev 3)

F.4D Seismic Coefficient Calculation (Rev 3)

F.4E Hydrostatic Uplift Analysis (Rev 3)

F.4F Settlement Analysis (Rev 3)

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9-27-2017

APPENDIX F.4

Geotechnical Calculations

APPENDIX F.4A

Shallow Translational Slope Stability Analysis (Rev 3)

AECOM

Job	**Scepter Waverly Landfill-East Phase**	Project No.	**60398526**	Sheet	**1**	Of **9**
Description	**Shallow Translational Slope**	Computed by	**NC**	Date	**1/15/17**	
	Stability Analysis (Rev 3)	Checked by	**SD**	Date	**09/10/17**	

I. PURPOSE

The purpose of this analysis is to evaluate the veneer stability of both the final cover system and bottom liner system at the proposed Scepter Waverly Class II Disposal Facility by evaluating shallow translational failure potential in those areas. There are no prescriptive standards related to slope stability in Tennessee's Solid Waste Processing and Disposal Regulations. These calculations are being provided pursuant to rule 1200-1-7-.04(2)(a)2 which states the facility must be located, designed, constructed, and maintained, and closed in such a manner as to minimize to the extent practicable the potential for releases of solid waste, solid waste constituents, or other potentially harmful material to the environment except in a manner authorized by state law.

II. SITE AND PROJECT DESCRIPTION

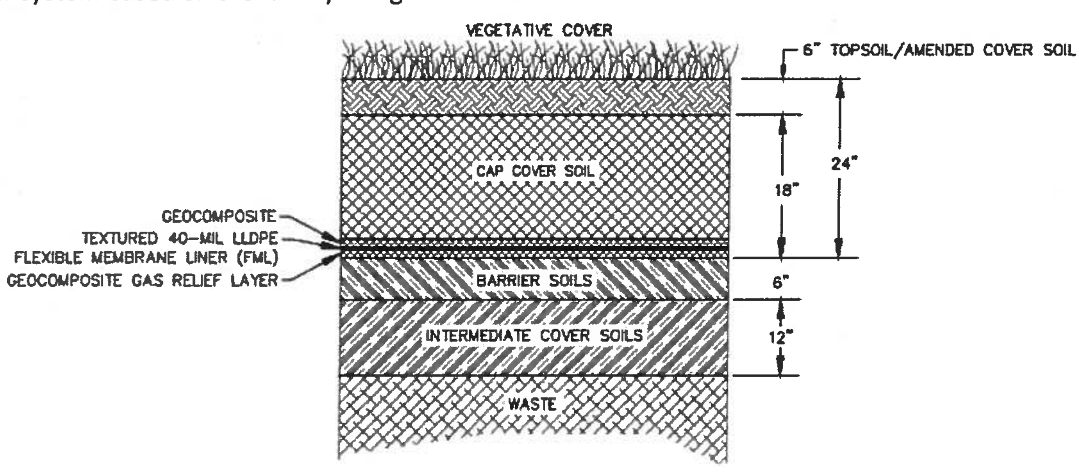
The shallow translational failure analysis was performed as part of the Part II Solid Waste Permit Application for the landfill facility for Scepter, Inc. in Waverly, Tennessee.

The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management.

The following sections summarize the methodology, assumptions, and results of the shallow translational failure analysis. For further detail on the specific calculations performed, refer to the corresponding data provided in the Attachments.

III. CONFIGURATION OF THE BOTTOM LINER AND FINAL COVER SYSTEM

Shallow translational failures were analyzed for the bottom liner system constructed on the excavation grades and final slopes. Conservative assumptions and parameters for the model were chosen for analysis including maximum grades based on the design. **Figure 1** depicts the bottom liner and final cover system based on the facility design.



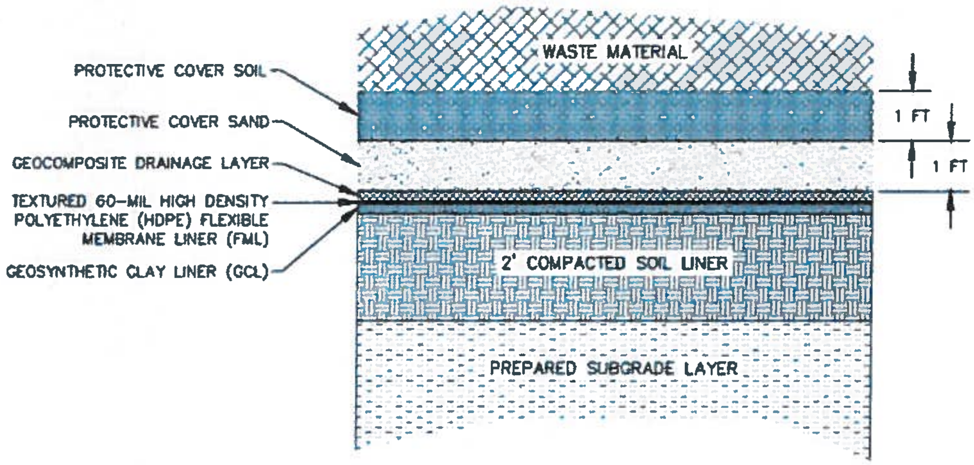


Figure 1. Scepter Waverly Bottom Liner and Final Cover Detail Design

IV. SHALLOW TRANSLATIONAL FAILURE ANALYSIS – METHODOLOGY

Analysis of the sliding potential of relatively thin cover soil layers (veneer) above both geosynthetic and natural soil liners, i.e. geomembranes (GM), geosynthetic clay liners (GCL) and compacted soil liners is important. This is because the underlying barrier materials generally represent a low interface shear strength boundary with respect to the soil placed above them.

The method used in this analysis follows the procedure outlined by Koerner and Soong (Koerner and Soong, 2005) and is performed by use of force equilibrium to balance the driving forces, due to gravity pulling on the cover soils, and the resistance to sliding, due to friction between the underlying subsurface and cover material. Resistance to sliding is also due in part to the toe support (passive wedge) located at the base of the sliding mass. This is illustrated in **Figure 2** below.

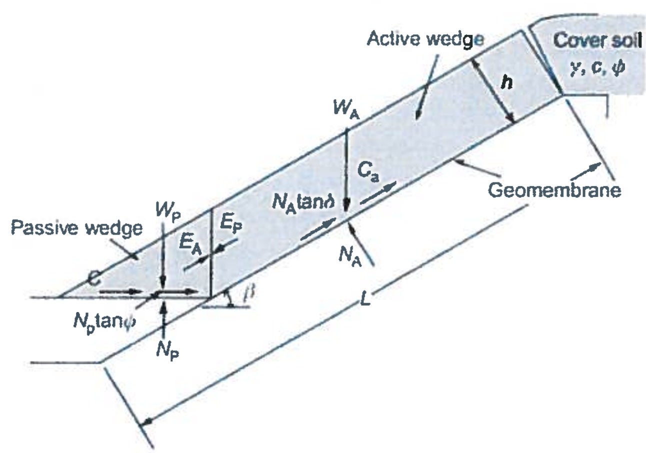


Figure 2. Conceptual Veneer Stability Analysis Cross/Free Body Diagram

Where,

W_A = Total Weight of the Active Wedge

W_P = Total Weight of the Passive Wedge

N_A = Effective Force Normal to the Failure Plane of the Active Wedge

N_P = Effective Force Normal to the Failure Plane of the Passive Wedge

γ = Unit Weight of the Cover Soil

h = Thickness of the Cover Soil

L = Length of Slope Measured Along the Geomembrane

β = Soil Slope Angle Beneath The Geomembrane

ϕ = Friction Angle Of The Cover Soil

δ = Interface Friction Angle Between Cover Soil and Geomembrane

C_A = Adhesive Force Between Active Wedge Cover Soil and Geomembrane

C = Cohesive Force Along The Failure Plane Of The Passive Wedge

c = Cohesion of the Cover Soil

E_A = Interwedge Force Acting on the Active Wedge from the Passive Wedge

E_P = Interwedge Force Acting on the Passive Wedge from the Active Wedge

FS = Factor of Safety Against Cover Soil Sliding on the Geomembrane

The shallow translational failure analysis is analyzed by fully satisfying the equilibrium of forces in the vertical and horizontal directions. By taking force summation parallel to the slope and comparing the resisting force with the driving or mobilizing force, a global factor of safety (FS) results:

$$FS = \frac{\Sigma\ Resisting\ Forces}{\Sigma\ Driving\ Forces}$$

As noted in the procedure contained in the Koerner and Soong paper, FS for veneer stability as depicted in **Figure 2** is determined by solving the following quadratic equation:

$$FS = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

Where,

$$a = (W_A - N_A cos\beta)cos\beta$$

$$b = -\begin{bmatrix} (W_A - N_A cos\beta)sin\beta tan\varphi \\ +(N_A tan\delta + C_a)sin\beta cos\beta \\ +sin\beta(C + W_P tan\phi) \end{bmatrix}$$

$$c = (N_A tan\delta + C_a)sin^2\beta tan\phi$$

When the calculated FS-value falls below 1.0, sliding of the cover soil on the geomembrane is to be anticipated.

For this analysis, the following conditions and factors of safety are recommended based on the anticipated probability and consequences of failure:

- Static Conditions (Peak Strength): FS ≥ 1.50
- Static Conditions (Residual Strength): FS ≥ 1.20
- Static Conditions (Full Drainage Layer): FS ≥ 1.10
- Static Conditions (Equipment Loads): FS ≥ 1.25
- Seismic Conditions: FS ≥ 1.00

The static/peak strength condition corresponds to a long term condition. Typically slope stability analyses require a static factor of safety between 1.4 and 2.0 (or higher if potential unknown conditions warrant it). A factor of safety of 1.5 is commonly used for landfill cap and liner system design for long term static conditions. Mobilizing residual strength in the engineered components is very unlikely, but the design was analyzed for residual strength conditions where applicable. Because this condition is unlikely to occur, a lower factor of safety is warranted. The design was also analyzed for static conditions under high seepage forces in the drainage layer where the drainage layer is at full capacity. This condition is also unlikely and temporary based on the frequency of the design storm used; therefore, a lower factor of safety is warranted. Other loads accounting for equipment on the side slope and seismic forces were analyzed representing temporary events warranting lower factors of safety.

Note that the analysis for equipment load only accounts for the weight of the vehicles and assumes very small and gradual acceleration and deceleration on the slope such that it can be neglected. It also assumes placement of the material beginning from the toe of slope progressing to the top.

V. SELECTION OF PARAMETERS

For the final cover and bottom liner system, the following assumptions and design parameters were used. Slope lengths and angles used in all analyses correspond to the maximum (i.e. worst case) values for each respective system.

a. Slope Geometry

The final cover and bottom liner system will have a maximum slope of 3H:1V with surface water swales benched into the waste mass every 30 foot increase in elevation. Based on the final cover design grades, the maximum slope length is approximately 110 feet. In the southeast corner of the proposed landfill, the bottom liner system has a long slope length of approximately 530 feet.

b. Layers

Layers and layer thicknesses for the cover system and liner system are anticipated as follows:

Table 1. Final Cover and Bottom Liner System Layer Summary

Thickness	Layer
Final Cover	
6 in	Vegetative Cover Soil
18 in	Cap Cover Soils
n/a	Geocomposite Drainage Layer
n/a	Flexible Membrane Liner
n/a	Geocomposite Gas Pressure Relief Layer
6 in	Protective Barrier Soils
12 in	Intermediate Cover Soils
Bottom Liner	
12 in	Protective Cover Layer
12 in	Protective Cover Layer Sand
n/a	Geocomposite Drainage Layer
n/a	Flexible Membrane Liner
n/a	Geosynthetic Clay Liner
24 in	Compacted Soil Layer

c. Critical Interfaces

The critical interfaces analyzed represent preferential pathways for mass sliding. Critical interfaces in the cover or liner system are typically between adjacent geosynthetic materials or between geosynthetic and oil materials. The geosynthetics are of negligible thickness so the depth to the failure surface does not require adjustment for the individual components when they are stacked.

For the final cover system, the following interfaces represent the critical interfaces analyzed:

- Protective cover layer to subsurface drainage geocomposite layer
- Subsurface drainage layer to flexible membrane liner
- Flexible membrane liner to gas pressure relief geocomposite layer
- Gas pressure relief geocomposite layer to subgrade soils

For the bottom liner system, the following interfaces represent the critical interfaces analyzed:

- Protective cover layer sand to geocomposite drainage layer
- Geocomposite drainage layer to flexible membrane liner
- Flexible membrane liner to geosynthetic clay liner
- Geosynthetic clay liner to compacted soil layer

For each system, a minimum shear strength envelop was determined for each condition. The most conservative envelope developed from the various conditions is to be used to determine material specifications for use in construction.

d. Material Parameters

The various material parameters used in the veneer analyses are tabulated below.

Table 2. Final Cover and Bottom Liner Variables

Parameter	Value	Comments
Final Cover		
Dry Unit Weight of Veg. Cover Soil, $\gamma_{D\text{-}VC}$	90 pcf	Assumed (typical of veg. material)
Moisture Content of Veg. Cover Soil, $w_{F\text{-}VC}$	26.0%	Assumed (typical of veg. material)
Dry Unit Weight of Slope Cover Soil, $\gamma_{D\text{-}CS}$	100 pcf	Typical unit weight of clay
Moisture Content of Slope Cover Soil, $w_{F\text{-}CS}$	24.0%	Assumed (typical value)
Specific Gravity of Slope Cover Soil, G_S	2.70	Assumed (typical of clay)
Friction Angle of Slope Cover Soil, ϕ	25.0°	Conservatively assumed cover soil
Cohesion of Slope Cover Soil, c	100 psf	Shear Strength parameters
Bottom Liner		
Dry Unit Weight of Prot. Cover Soil, $\gamma_{D\text{-}PC}$	110 pcf	Typical unit weight of silty material
Moisture Content of Prot. Cover Soil, $w_{F\text{-}PC}$	26.0%	Assumed (typical value)
Specific Gravity of Prot. Cover Soil, $G_{S\text{-}PC}$	2.68	Assumed (typical of silt)
Friction Angle of Prot. Cover Soil, ϕ_{PC}	25.0°	Conservatively assumed for silt
Cohesion of Slope Cover Soil, c_{PC}	100 psf	Strength parameters
Dry Unit Weight of Prot. Cover Soil Sand, $\gamma_{D\text{-}PCS}$	105 pcf	Typical unit weight of sand
Moisture Content of Prot. Cover Soil Sand, $w_{F\text{-}PCS}$	24.0%	Assumed (typical value)
Specific Gravity of Prot. Cover Soil Sand, $G_{S\text{-}PCS}$	2.65	Assumed (typical of sand)
Friction Angle of Prot. Cover Soil Sand, ϕ_{PCS}	30.0°	Conservatively assumed for sand
Cohesion of Slope Cover Soil Sand, c_{PCS}	0 psf	Strength parameters
Equipment Loads		
Weight of Construction Equip., W_b	39,918 lbs	CAT D6 LGP Dozer
Length of Equip. Track, w	10.2 ft	CAT D6 LGP Dozer
Width of Equip. Track, b	2.8 ft	CAT D6 LGP Dozer
Seismic Loads		
Seismic Coefficient (Cover)	0.266g	Seismic Coefficient Calculation
Seismic Coefficient (Liner)	0.133g	Seismic Coefficient Calculation

VI. RESULTS OF ANALYSIS

The analysis consists of finding strength parameters (friction angle, ϕ, and cohesion, c) for a given interface to meet required factors of safety against translational failure. In the case of geosynthetic interfaces, the term adhesion is used instead of cohesion. Cohesion strictly relates to clay particle interaction, whereas adhesion refers to the physical interaction of geosynthetic surfaces. Mathematically, these terms are interchangeable.

The results of the analysis are summarized below. Detailed calculations are included as **Attachment A**. The minimum peak and residual shear strength parameters for each condition are included in graphical form as Figures A.1, A.2, B.1 and B.2 in **Attachment B**.

a. Final Cover System Shear Strength Parameters

The minimum required shear strength parameters for the final cover system are as follows assuming cohesion/adhesion is equal to zero. Additional minimum values where cohesion/adhesion are non-zero are included in **Attachment B**.

Table 3. Final Cover Minimum Calculated Required Friction Angles

Condition	Peak Interface Friction, ϕ (degrees)*	Factor of Safety
Static	25.8	1.50
Static (Full Drainage Layer)	20.1	1.10
Static (Equipment Loads)	21.9	1.25
Seismic	31.5	1.00

*Calculations of friction angles assume interface adhesion (c) is equal to zero.

The critical interface condition was found to be the seismic condition. Therefore, the minimum friction angle needed to be met for the cap system interfaces is 28.0 degrees.

b. Bottom Liner System Shear Strength Parameters

The minimum required shear strength parameters for the bottom liner system are as follows assuming cohesion/adhesion is equal to zero. Additional minimum values where cohesion/adhesion are non-zero are included in **Attachment B**.

Table 4. Bottom Liner Minimum Calculated Required Friction Angles

Condition	Peak Interface Friction, ϕ (degrees)*	Factor of Safety
Static	26.4	1.50
Static (Full Drainage Layer)	22.8	1.10
Static (Equipment Loads)	22.4	1.25
Seismic	25.5	1.00

*Calculations of friction angles assume interface adhesion (c) is equal to zero.

The critical condition for the bottom liner system interface was found to be the seismic condition. Therefore, the minimum friction angle needed to be met for the bottom liner system interfaces is 28.0 degrees. Preconstruction testing with actual materials is anticipated to verify that the materials used exhibit interface properties above the minimum shear strength envelope.

c. Residual Strength

The acceptable material specifications are anticipated to be based on peak strength of the materials. However, should the cover or liner system be temporarily acted upon by an outside force that causes the post peak strength to be mobilized, there may be some displacement of the cap system. Movement will occur along the interface with the lowest peak strength. This will then mobilize the residual/large-displacement strength of that particular interface. If residual strength for the interface with the lowest peak strength is above the minimum shear strength envelope as depicted in **Attachment B**, the cap system is anticipated to stabilize after the temporary loading condition ends.

Preconstruction testing with actual materials should be performed to verify that the materials used exhibit interface properties above the minimum shear strength envelope.

The minimum shear strength parameters assuming residual strength is mobilized for the final cover and bottom liner system are as follows assuming cohesion/adhesion is equal to zero. Additional minimum values where cohesion/adhesion are non-zero are included in **Attachment B.**

Table 5. Minimum Calculated Residual Friction Angles

Condition	Peak Interface Friction, ϕ (degrees)*	Factor of Safety
Final Cover Static	19.2	1.20
Bottom Liner Static	19.9	1.20

*Calculations of friction angles assume interface adhesion (c) is equal to zero.

VII. REFERENCES

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VIII. ATTACHMENTS

Attachment A	Veneer Stability (Liner and Cover)
Attachment B	Minimum Shear Strength Envelope

AECOM	Scepter	JOB	60398526		
1000 Corporate Centre Dr., Suite 250		SHT NO	1	OF	11
Franklin, TN 37067	VENEER STABILITY (LINER)	CALC BY	MW	DATE	12/04/16
Tel. (615) 771 2480	East Phase Disposal Facility	CHK BY	NSG	DATE	12/6/16
Fax (615) 771 2459		SCALE	NA		

Objective:

Determine the veneer stability of the bottom liner system at the Scepter Waverly Landfill Facility for the Part II Permit.

Method:

Use methods outlined in the paper by Koerner and Soong, *Analysis and Design of Veneer Cover Soils* published in **Geosynthetics International**, 2005, 12, No.1.

Procedure:

Determine the static stability of the veneer bottom liner system to determine the minimum required interface friction angle for all engineered components of the liner system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters.

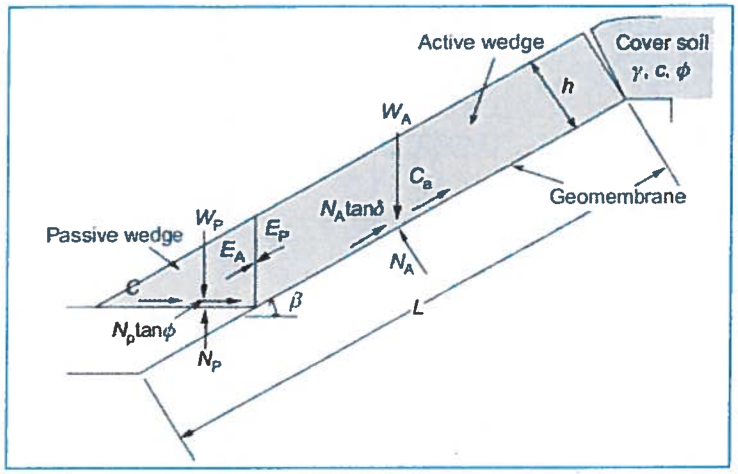

Figure 1. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil

Determine: Static factor of safety for liner system based on gravitational forces only and peak strength.

Assumptions:

1. No geosynthetic reinforcements
2. No interface adhesion for geosynthetic components.
3. No tension allowed in geosynthetics
4. Minimum cohesion for multilayered systems

Prot. Cover Layer (PC)/Prot. Cover Layer Sand (PCS) and Slope Parameters	PC	PCS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	110.0	105.0	106.3	pcf	Assumed values based on soil type
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed conservative
Avg Field Unit Wt (γ)	-	-	132.3	pcf	
Reference Stress	-	-	**251.0**	psf	
Min. Friction Angle (ϕ)	25.0	30.0	28.8	deg	Assumed conservative
Min. Cohesion (c)	100.0	0.0	0.0	psf	Assumed conservative
Slope Angle Beneath the Geom. (β)	-	-	18.43	deg	Design - see Sheet 07
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	167.00	ft	Design - see Sheet 07
Lng. of Slp. Meas. Along Geom. (L)	-	-	528.24	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	26.3	deg	Minimum values to achieve factor of safety
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	1.50		Min. req. FS

AECOM	Scepter	JOB	60398526			
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Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) = 137{,}990 \text{ lbs/ft}$$

$$N_A = W_A \cos\beta = 130{,}913 \text{ lbs/ft}$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) = - \text{ lbs/ft}$$

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)} = 491 \text{ lbs/ft}$$

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} = 882 \text{ lbs/ft}$$

$$C = \frac{ch}{\sin\beta} = - \text{ lbs/ft}$$

$$E_P = \frac{C + W_P\tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} = 387 \text{ lbs/ft}$$

$$N_P = W_P + E_P\sin\beta = 1{,}005 \text{ lbs/ft}$$

Static Factor of Safety (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A\cos\beta)\cos\beta = 13{,}084 \text{ lbs/ft}$$

$$b = -\begin{bmatrix}(W_A - N_A\cos\beta)\sin\beta\tan\phi \\ +(N_A\tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P\tan\phi)\end{bmatrix} = (21{,}951) \text{ lbs/ft}$$

$$c = (N_A\tan\delta + C_a)\sin^2\beta\tan\phi = 3{,}548 \text{ lbs/ft}$$

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a} = 1.50$$

Results Have Converged		
FS_R =	1.50	
FS_A =	1.50	
Min. Peak δ	26.3	deg
Min. Peak c_a	0.0	psf

AECOM	Scepter	JOB	60398526		
1000 Corporate Centre Dr., Suite 250		SHT NO	3	OF	11
Franklin, TN 37067	VENEER STABILITY (LINER)	CALC BY	MW	DATE	12/4/16
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Determine: Static factor of safety for liner system based on additional seepage forces.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for seepage forces in drainage layer as noted in **Figure 2**.

Assumptions:

In addition to the static case assumptions:
1. Seepage is parallel to slope
2. The drainage layer is sized such that liquid not build up beyond the thickness of the drainage layer.
3. Drainage layer has adequate capacity to handle maximum surface water flow
4. If geocomposite is used - it is less than 0.75 inches thick.
5. Max accumulation of up to 1 foot head on top of FML barrier to account for drainage aggregate in lieu of geocomposite.

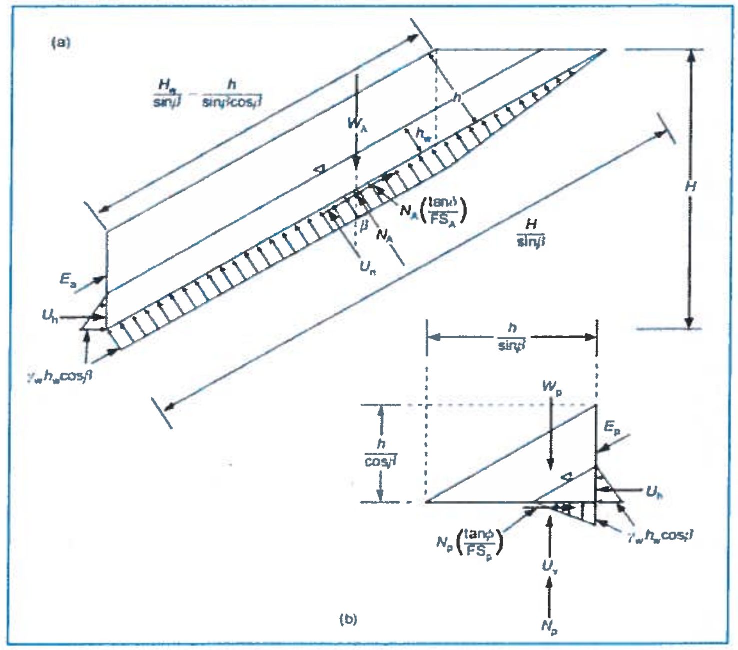

Figure 2. Limit equilibrium forces involved in finite-length slope of uniform cover soil with parallel-to-slope seepage build-up: (a) active wedge; (b) passive wedge

Prot. Cover Layer (PC)/Prot. Cover Layer Sand (PCS) and Slope Parameters	PC	PCS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	110.0	105.0	106.3	pcf	Assumed values based on soil type
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed conservative
Avg Field Unit Wt (γ)	-	-	132.3	pcf	
Specfic Gravity of the (G_S)	2.68	2.65	2.66		**Assumed conservative**
Unit Weight of Water (γ_W)	-	-	62.4	pcf	
Saturated Unit Weight (γ_{SAT})	-	-	128.7	pcf	
Min. Friction Angle (ϕ)	25.0	30.0	28.8	deg	Assumed conservative
Min. Cohesion (c)	100.0	0.0	0.0	psf	Assumed conservative
Slope Angle Beneath the Geom. (β)	-	-	18.4	degrees	Design - see Sheet 07
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	167.00	ft	Design - see Sheet 07
Lng. of Slp. Meas. Along Geom. (L)	-	-	528.2	ft	
Depth of Water in DL (h_W)	-	-	0.08	ft	**> Max. thickness of DL (conservative)**
Inter. Frict. Angle for DL & Geom. (δ)	-	-	20.4	deg	Minimum values to achieve factor of safety
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	1.10		**Min. req. FS for temporary conditions**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A), resultant of the pore pressures acting on the interwedge surfaces (U_h), resultant of the pore pressures acting perpendicular to the slope (U_n), the effective force normal to the failure plane of the active wedge (N_A), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \frac{\gamma_D (h - h_W)[2H_L \cos\beta - (h + h_W)]}{\sin 2\beta} + \frac{\gamma_{SAT}(h_W)(2H_L \cos\beta - h_W)}{\sin 2\beta}$$ = **112,489 lbs/ft**

$$U_h = \frac{\gamma_w h_W^2}{2}$$ = **0.20 lbs/ft**

$$U_n = \frac{\gamma_w (h_w)(\cos\beta)(2H_L \cos\beta - h_w)}{\sin 2\beta}$$ = **2,501 lbs/ft**

$$N_A = W_A \cos\beta + U_h \sin\beta - U_n$$ = **104,219 lbs/ft**

$$E_A = W_A \sin\beta - U_h \cos\beta - \frac{N_A \sin\delta}{(FS)}$$ = **2,538 lbs/ft**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P), resultant of the vertical pore pressures acting on the passive wedge (U_V), and the interwedge force acting on the pass wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma_D (h^2 - h_w^2) + \gamma_{SAT} h_w^2}{\sin 2\beta}$$ = **708.7 lbs/ft**

$$U_V = U_h \cot\beta$$ = **0.6 lbs/ft**

$$E_P = \frac{U_h (FS) - (W_P - U_V \tan\phi)}{\sin\beta \tan\phi - \cos\beta (FS)}$$ = **814 lbs/ft**

Static Factor of Safety w/ Seepage Forces (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = W_A \sin\beta \cos\beta - U_h \cos^2\beta + U_h$$ = **33739 lbs/ft**

$$b = -W_A \sin^2\beta \tan\phi + U_h \sin\beta \cos\beta \tan\phi - N_A \cos\beta \tan\delta - (W_P - U_V)\tan\phi$$ = **(43,327.3) lbs/ft**

$$c = N_A \sin\beta \tan\delta \tan\phi$$ = **6,722 lbs/ft**

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$ = **1.10**

Allow Exceeds Req'd - OK

FS_R =	1.10	
FS_A =	1.10	
Min. Peak δ	20.4	deg
Min. Peak c_a	0.0	psf

Determine: Static factor of safety for liner system based on additional equipment loads.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for equipment loads (W_b) as final cover is placed as noted in Figure 3.

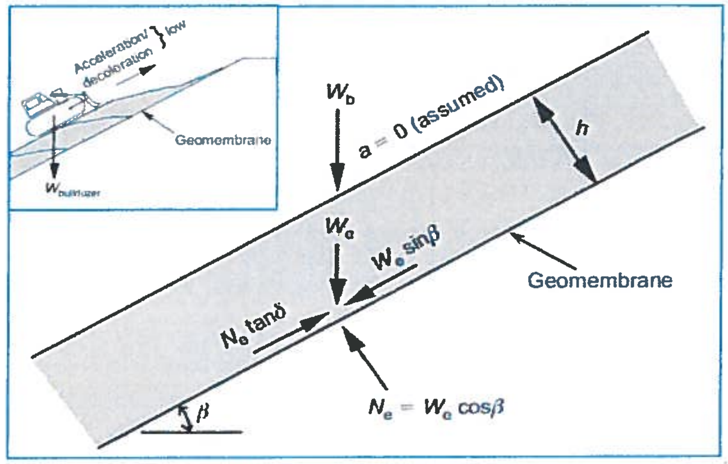

Figure 3. Additional load due to construction equipment moving on cover soil.

Assumptions:

In addition to the static case assumptions:
1. The equipment pushes material up slope leaving a toe buttress behind.
2. The equipment accelerates slowly with no sudden starts or turns to minimize additional loads besides the weight of the machine.

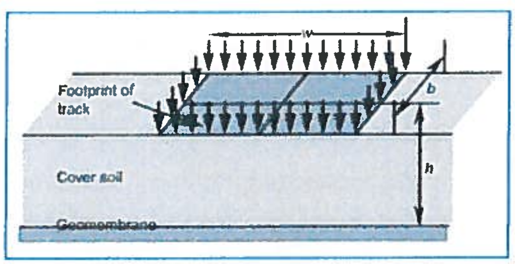

Figure 4. Illustration of stress distribution from overlying equipment.

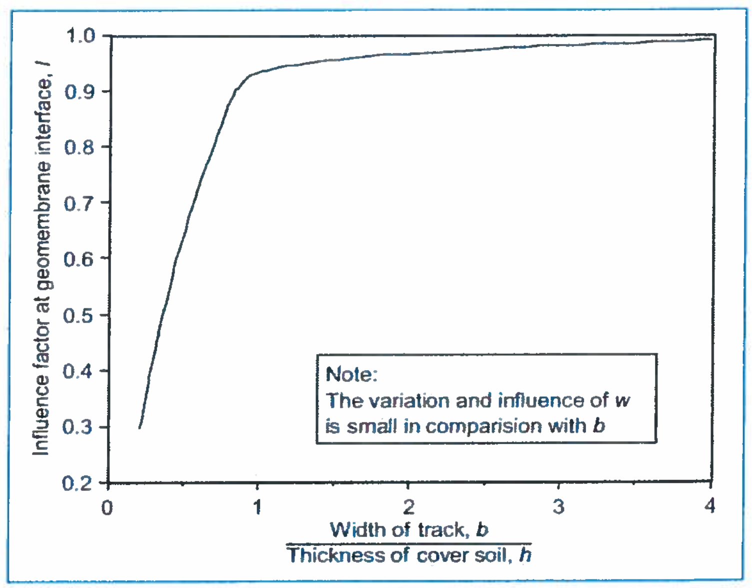

Figure 5. Values of influence factor I to dissipate surface force through cover soil to geomembrane interface (after Poulos and Davis 1974)

Equipment Parameters			Source
Equiv. Equipment Load per Unit Width (W_e)	6797	lbs/ft	W_e = qwl
Influence Factor at the Geom. Interface (I)	0.95		See Figure 5 above.
Track Width to Cover Soil Thickness Ratio (b/h)	1.40		
Distributed Equipment Load (q)	699	psf	$q = W_b/(2 \times w \times b)$
Weight of Equipment (W_b)	39,918	lbs	**Typical weight of CAT D6 dozer**
Length of Equipment Track (w)	10.20	ft	
Width of Equipment Track (b)	2.80	ft	**Typical track dimensions of CAT D6 dozer**

AECOM	Scepter	JOB	60398526		
1000 Corporate Centre Dr., Suite 250		SHT NO	6	OF	11
Franklin, TN 37067	VENEER STABILITY (LINER)	CALC BY	MW	DATE	12/4/16
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Prot. Cover Layer (PC)/Prot. Cover Layer Sand (PCS) and Slope Parameters

	PC	PCS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	110.0	105.0	106.3	pcf	Assumed values based on soil type
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed conservative
Avg Field Unit Wt (γ)	-	-	132.3	pcf	
Min. Friction Angle (ϕ)	25.0	30.0	28.8	deg	Assumed conservative
Min. Cohesion (c)	100.0	0.0	0.0	psf	Assumed conservative
Slope Angle Beneath the Geom. (β)	-	-	18.4	deg	Design - see Sheet 07
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	167.00	ft	Design - see Sheet 07
Lng. of Slp. Meas. Along Geom. (L)	-	-	528.24	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	22.4	deg	Minimum values to achieve factor of safety
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	1.25		**Min. req. FS for temporary conditions**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) + W_e = \mathbf{144{,}787 \ lbs/ft}$$

$$N_A = W_A \cos\beta = \mathbf{137{,}361 \ lbs/ft}$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) = \mathbf{- \ lbs/ft}$$

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)} = \mathbf{481 \ lbs/ft}$$

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} = \mathbf{882 \ lbs/ft}$$

$$C = \frac{ch}{\sin\ \beta} = \mathbf{- \ lbs/ft}$$

$$E_P = \frac{C + W_P\tan\phi}{\cos\beta(FS) - \sin\ \beta\tan\phi} = \mathbf{478 \ lbs/ft}$$

$$N_P = W_P + E_P\sin\ \beta = \mathbf{1{,}033 \ lbs/ft}$$

Static Factor of Safety w/ Equipment Load (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A \cos\beta)\cos\beta \quad = \quad 13{,}729 \text{ lbs/ft}$$

$$b = -\begin{bmatrix} (W_A - N_A\cos\beta)\sin\beta\tan\phi \\ +(N_A\tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P\tan\phi) \end{bmatrix} \quad = \quad (19{,}644) \text{ lbs/ft}$$

$$c = (N_A\tan\delta + C_a)\sin^2\beta\tan\phi \quad = \quad 3104 \text{ lbs/ft}$$

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \quad = \quad 1.25$$

Allow Exceeds Req'd - OK		
FS_R =	1.25	
FS_A =	1.25	
Min. Peak δ	22.4	deg
Min. Peak c_a	0.0	psf

AECOM	Scepter	JOB	60398526		
1000 Corporate Centre Dr., Suite 250		SHT NO	8	OF	11
Franklin, TN 37067	VENEER STABILITY (LINER)	CALC BY	MW	DATE	12/4/16
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Determine: Static factor of safety for liner system based on additional seismic loads.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for seismic loads (C_s) as noted in Figure 6.

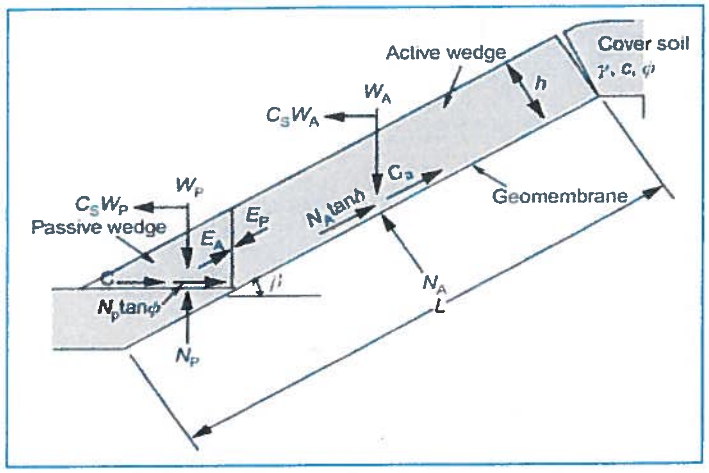

Figure 6. Limit equilibrium forces involved in pseudo-static analysis using average seismic coefficient.

Assumptions:

In addition to the static case assumptions:
1. Seismic force acts on the centroid of the cover soil.
2. Seismic force is horizontal.
3. Deformation analysis not required.

Prot. Cover Layer (PC)/Prot. Cover Layer Sand (PCS) and Slope Parameters	PC	PCS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	110.0	105.0	106.3	pcf	Assumed values based on soil type
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed conservative
Avg Field Unit Wt (γ)	-	-	132.3	pcf	
Min. Friction Angle (ϕ)	25.0	30.0	28.8	deg	Assumed conservative
Min. Cohesion (c)	100.0	0.0	0.0	psf	Assumed conservative
Slope Angle Beneath the Geom. (β)	-	-	18.4	degrees	Design - see Sheet 07
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	167.00	ft	Design - see Sheet 07
Lng. of Slp. Meas. Along Geom. (L)	-	-	528.24	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	25.6	deg	Minimum values to achieve factor of safety
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Avg. Seismic Coeffcient (C_S or K_S)	-	-	0.133	%g	**Seismic Coefficient Calcs**
Required Factor of Safety (FS_R)	-	-	1.00		**Min. req. FS for seismic conditions**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) =$$ **137,990 lbs/ft**

$$N_A = W_A \cos\beta =$$ **130,913 lbs/ft**

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) =$$ **- lbs/ft**

$$E_A = \frac{(FS)(C_S W_A + N_A \sin\beta)}{(FS)\cos\beta} - \frac{(N_A \tan\delta + C_a)\cos\beta}{(FS)\cos\beta} =$$ **247 lbs/ft**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} =$$ **882 lbs/ft**

$$C = \frac{ch}{\sin\beta} =$$ **- lbs/ft**

$$E_P = \frac{C + W_P \tan\phi - C_S W_P (FS)}{\cos\beta (FS) - \sin\beta \tan\phi} =$$ **473 lbs/ft**

$$N_P = W_P + E_P \sin\beta =$$ **1,032 lbs/ft**

Seismic Factor of Safety (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (C_S W_A + N_A \sin\beta)\cos\beta + C_S W_P \cos\beta =$$ **56,787 lbs/ft**

$$b = -\begin{bmatrix} (C_S W_A + N_A \sin\beta)\sin\beta \tan\phi \\ + (N_A \tan\delta + C_a)\cos^2\beta \\ + (C + W_P \tan\phi)\cos\beta \end{bmatrix} =$$ **(67,274) lbs/ft**

$$c = (N_A \tan\delta + C_a)\cos\beta \sin\beta \tan\phi =$$ **10321 lbs/ft**

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a} =$$ **1.00**

Allow Exceeds Req'd - OK		
FS_R =	1.00	
FS_A =	1.00	
Min. Peak δ	25.6	deg
Min. Peak c_a	0.0	psf

AECOM	Scepter	JOB	60398526		
1000 Corporate Centre Dr., Suite 250		SHT NO	10	OF	11
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Determine: Static factor of safety for liner system based on residual strength.

Procedure:

Determine the static stability of the veneer cover system to evaluate the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters.

Assumptions:

No additional assumptions

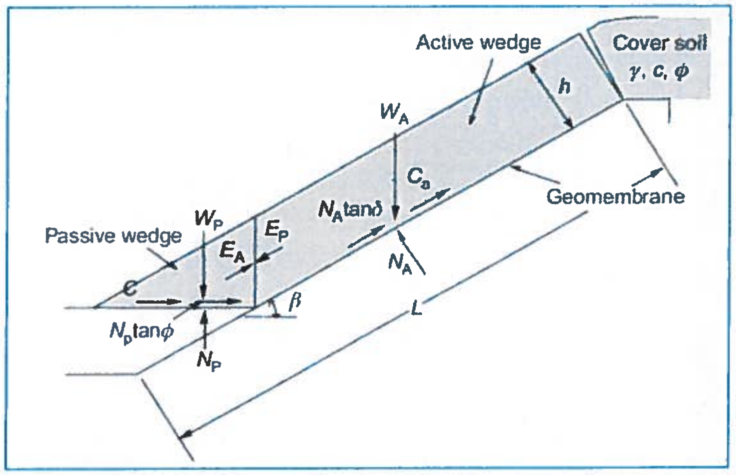

Figure 7. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil

Prot. Cover Layer (PC)/Prot. Cover Layer Sand (PCS) and Slope Parameters	PC	PCS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	110.0	105.0	106.3	pcf	Assumed values based on soil type
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed conservative
Avg Field Unit Wt (γ)	-	-	132.3	pcf	
Reference Stress	-	-	**251.0**	**psf**	
Min. Friction Angle (ϕ)	25.0	30.0	28.8	deg	Assumed conservative
Min. Cohesion (c)	100.0	0.0	0.0	psf	Assumed conservative
Slope Angle Beneath the Geom. (β)	-	-	18.4	deg	Design - see Sheet 07
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	167.00	ft	
Lng. of Slp. Meas. Along Geom. (L)	-	-	528.24	ft	Design - see Sheet 07
Inter. Frict. Angle for DL & Geom. (δ)	-	-	**21.6**	deg	**Minimum values to achieve factor of safety**
Adhesion for DL & Geom. (c_a)	-	-	**0.0**	psf	
Required Factor of Safety (FS_R)	-	-	**1.20**		**Min. req. FS**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right)$$ = **137,990 lbs/ft**

$$N_A = W_A \cos\beta$$ = **130,913 lbs/ft**

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right)$$ = **- lbs/ft**

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)}$$ = **432 lbs/ft**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} = 882 \text{ lbs/ft}$$

$$C = \frac{ch}{\sin \beta} = - \text{ lbs/ft}$$

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} = 501 \text{ lbs/ft}$$

$$N_P = W_P + E_P \sin\beta = 1{,}041 \text{ lbs/ft}$$

Static Factor of Safety for Residual Strength (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A \cos\beta)\cos\beta = 13{,}084 \text{ lbs/ft}$$

$$b = -\begin{bmatrix} (W_A - N_A\cos\beta)\sin\beta\tan\phi \\ +(N_A\tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P\tan\phi) \end{bmatrix} = (18{,}091) \text{ lbs/ft}$$

$$c = (N_A \tan\delta + C_a)\sin^2\beta\tan\phi = 2{,}842 \text{ lbs/ft}$$

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a} = 1.20$$

Results Have Converged

FS_R =	1.20	
FS_A =	1.20	
Min. Resid. δ	21.6	deg
Min. Resid. c_a	0.0	psf

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VENEER STABILITY (FINAL COVER)
East Phase Disposal Facility

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SHT NO	1	OF	11
CALC BY	MW	DATE	12/04/16
CHK BY	NSG	DATE	12/6/16
SCALE	NA		

Objective:

Determine the veneer stability of the cover system at the Scepter Waverly Landfill Facility for the Part II Permit.

Method:

Use methods outlined in the paper by Koerner and Soong, *Analysis and Design of Veneer Cover Soils* published in **Geosynthetics International,** 2005, 12, No.1.

Procedure:

Determine the static stability of the veneer cover system to evaluate the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in **Figure 1** and the required factor of safety (FS) then solve for minimum interface shear strength parameters.

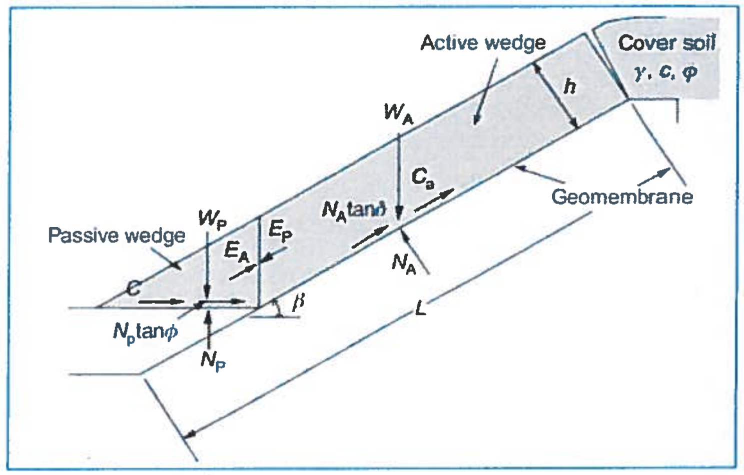

Figure 1. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil

Determine: Static factor of safety for cover system based on gravitational forces only and peak strength.

Assumptions:

1. No geosythetic reinforcements.
2. No interface adhesion for geosynthetic components.
3. No tension allowed in geosynthetics.
4. Minimum cohesion for multilayered systems.

Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	90.0	110.0	105.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	130.7	pcf	
Reference Stress	-	-	**248.0**	**psf**	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	18.43	deg	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	110.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	347.94	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	25.9	deg	Minimum values to achieve Safety Factor
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	1.50		Min. req'd FS for long term conditions

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right)$$ = **89,228 lbs/ft**

$$N_A = W_A\cos\beta$$ = **84,652 lbs/ft**

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right)$$ = **- lbs/ft**

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)}$$ = **806 lbs/ft**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta}$$ = **872 lbs/ft**

$$C = \frac{ch}{\sin\beta}$$ = **633 lbs/ft**

$$E_P = \frac{C + W_P\tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi}$$ = **815 lbs/ft**

$$N_P = W_P + E_P\sin\beta$$ = **1,129 lbs/ft**

Static Factor of Safety (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A\cos\beta)\cos\beta$$ = **8,461 lbs/ft**

$$b = -\begin{bmatrix}(W_A - N_A\cos\beta)\sin\beta\tan\phi \\ +(N_A\tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P\tan\phi)\end{bmatrix}$$ = **(13,972) lbs/ft**

$$c = (N_A\tan\delta + C_a)\sin^2\beta\tan\phi$$ = **1,916 lbs/ft**

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$ = **1.50**

Results Have Converged

FS_R =	1.50	
FS_A =	1.50	
Min. Peak δ	25.9	deg
Min. Peak c_a	0.0	psf

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Determine: Static factor of safety for cover system based on additional seepage forces.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for seepage forces in drainage layer as noted in **Figure 2**.

Assumptions:

In addition to the static case assumptions:
1. Seepage is parallel to slope
2. The drainage layer is sized such that liquid not build up beyond the thickness of the drainage layer.
3. Drainage layer has adequate capacity to handle maximum surface water flow
4. If geocomposite is used - it is less than 0.75 inches thick.
5. Max accumulation of up to 1 foot head on top of FML barrier to account for drainage aggregate in lieu of geocomposite.

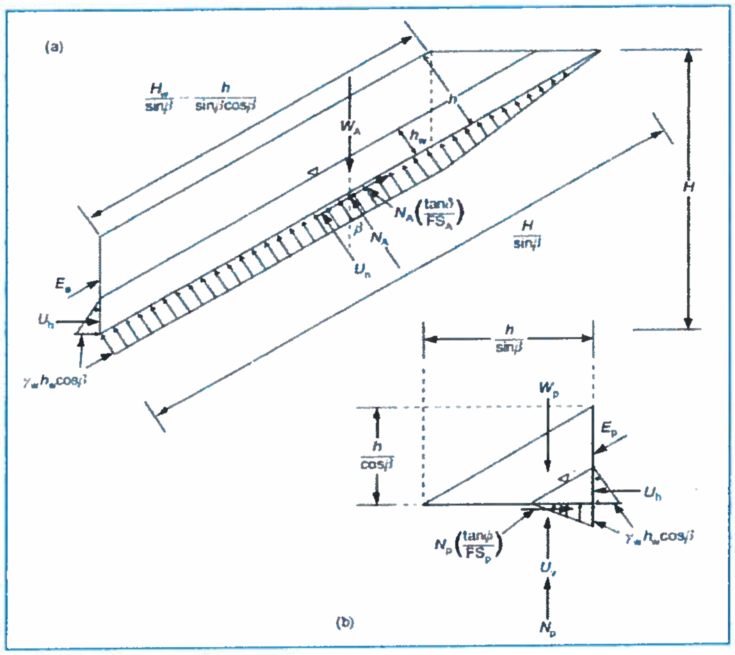

Figure 2. Limit equilibrium forces involved in finite-length slope of uniform cover soil with parallel-to-slope seepage build-up: (a) active wedge; (b) passive wedge

Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	90.0	110.0	105.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	130.7	pcf	
Specfic Gravity of the (G_S)	2.74	2.70	2.71		**Assumed conservative**
Unit Weight of Water (γ_W)	-	-	62.4	pcf	
Saturated Unit Weight (γ_{SAT})	-	-	128.7	pcf	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	18.43	degrees	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	110.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	347.9	ft	
Depth of Water in DL (h_W)	-	-	0.08	ft	**Max. thickness of DL (conservative)**
Inter. Frict. Angle for DL & Geom. (δ)	-	-	20.2	deg	Minimum values to achieve Safety Factor
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	1.10		**Min. req'd FS for temporary conditions**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A), resultant of the pore pressures acting on the interwedge surfaces (U_h), resultant of the pore pressures acting perpendicular to the slope (U_n), the effective force normal to the failure plane of the active wedge (N_A), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \frac{\gamma_D (h - h_W)[2H_L \cos\beta - (h + h_W)]}{\sin 2\beta} + \frac{\gamma_{SAT}(h_W)(2H_L \cos\beta - h_W)}{\sin 2\beta}$$ = **73,026 lbs/ft**

$$U_h = \frac{\gamma_w h_W^2}{2}$$ = **0.20 lbs/ft**

$$U_n = \frac{\gamma_w (h_w)(\cos\beta)(2H_L \cos\beta - h_w)}{\sin 2\beta}$$ = **1,647 lbs/ft**

$$N_A = W_A \cos\beta + U_h \sin\beta - U_n$$ = **67,633 lbs/ft**

$$E_A = W_A \sin\beta - U_h \cos\beta - \frac{N_A \sin\delta}{(FS)}$$ = **1,856 lbs/ft**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P), resultant of the vertical pore pressures acting on the passive wedge (U_V), and the interwedge force acting on the pass wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma_D (h^2 - h_w^2) + \gamma_{SAT} h_w^2}{\sin 2\beta}$$ = **700.4 lbs/ft**

$$U_V = U_h \cot\beta$$ = **0.6 lbs/ft**

$$E_P = \frac{U_h (FS) - (W_P - U_V \tan\phi)}{\sin\beta \tan\phi - \cos\beta (FS)}$$ = **781 lbs/ft**

Static Factor of Safety w/ Seepage Forces (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = W_A \sin\beta \cos\beta - U_h \cos^2\beta + U_h$$ = **21,903 lbs/ft**

$$b = -W_A \sin^2\beta \tan\phi + U_h \sin\beta \cos\beta \tan\phi - N_A \cos\beta \tan\delta - (W_P - U_V)\tan\phi$$ = **(27,337.6) lbs/ft**

$$c = N_A \sin\beta \tan\delta \tan\phi$$ = **3,668 lbs/ft**

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$ = **1.10**

Allow Exceeds Req'd - OK		
FS_R =	1.10	
FS_A =	1.10	
Min. Peak δ	20.2	deg
Min. Peak c_a	0.0	psf

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Determine: Static factor of safety for cover system based on additional equipment loads.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for equipment loads (W_b) as final cover is placed as noted in Figure 3.

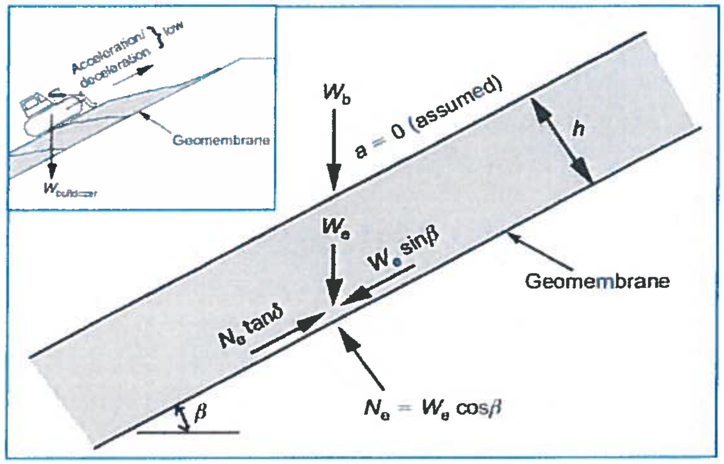

Figure 3. Additional load due to construction equipment moving on cover soil.

Assumptions:

In addition to the static case assumptions:
1. The equipment pushes material up slope leaving a toe buttress behind.
2. The equipment accelerates slowly with no sudden starts or turns to minimize additional loads besides the weight of the machine.

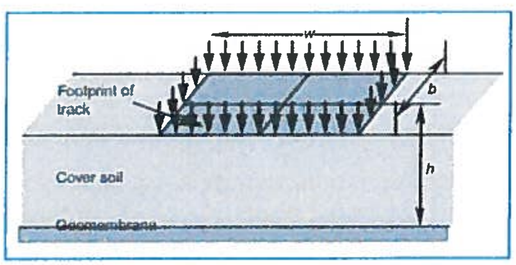

Figure 4. Illustration of stress distribution from overlying equipment.

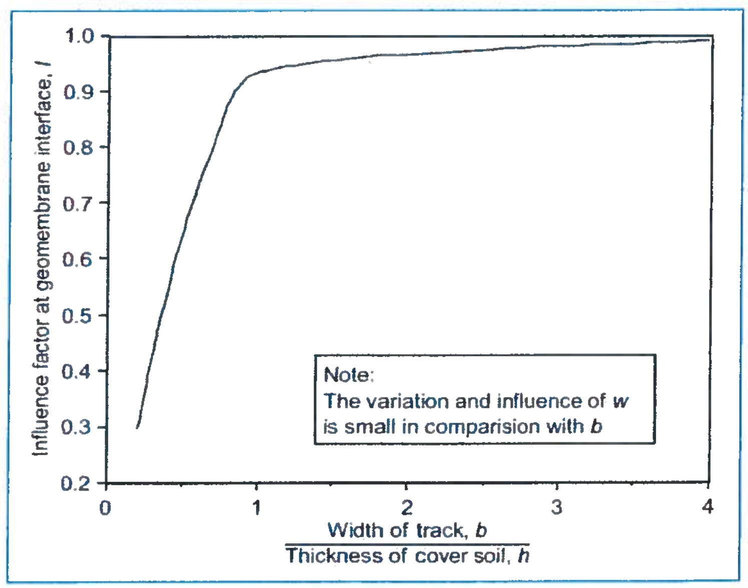

Figure 5. Values of influence factor I to dissipate surface force through cover soil to geomembrane interface (after Poulos and Davis 1974)

Equipment Parameters			Source
Equiv. Equipment Load per Unit Width (W_e)	6797	lbs/ft	W_e = qwl
Influence Factor at the Geom. Interface (I)	0.95		See Figure 5 above.
Track Width to Cover Soil Thickness Ratio (b/h)	1.40		
Distributed Equipment Load (q)	699	psf	q = $W_b/(2 \times w \times b)$
Weight of Equipment (W_b)	**39,918**	lbs	**Typical weight of CAT D6 dozer**
Length of Equipment Track (w)	**10.20**	ft	
Width of Equipment Track (b)	**2.80**	ft	**Typical track dimensions of CAT D6 dozer**

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SHT NO	6	OF	11
CALC BY	MW	DATE	12/4/16
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Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters

	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	90.0	110.0	105.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	130.7	pcf	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	18.43	deg	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	110.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	347.94	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	22.0	deg	Minimum values to achieve Safety Factor
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Required Factor of Safety (FS_R)	-	-	1.25		**Min. req'd FS for temporary conditions**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right) + W_e = \mathbf{96{,}025\ lbs/ft}$$

$$N_A = W_A \cos\beta = \mathbf{91{,}100\ lbs/ft}$$

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right) = \mathbf{-\ lbs/ft}$$

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)} = \mathbf{912\ lbs/ft}$$

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} = \mathbf{872\ lbs/ft}$$

$$C = \frac{ch}{\sin\ \beta} = \mathbf{633\ lbs/ft}$$

$$E_P = \frac{C + W_P\tan\phi}{\cos\beta(FS) - \sin\ \beta\tan\phi} = \mathbf{1{,}001\ lbs/ft}$$

$$N_P = W_P + E_P\sin\ \beta = \mathbf{1{,}188\ lbs/ft}$$

Static Factor of Safety w/ Equipment Load (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A \cos\beta)\cos\beta$$ = **9,105 lbs/ft**

$$b = -\begin{bmatrix} (W_A - N_A \cos\beta)\sin\beta \tan\phi \\ +(N_A \tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P \tan\phi) \end{bmatrix}$$ = **(12,783) lbs/ft**

$$c = (N_A \tan\delta + C_a)\sin^2\beta \tan\phi$$ = **1715 lbs/ft**

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$ = **1.25**

Allow Exceeds Req'd - OK		
FS_R =	1.25	
FS_A =	1.25	
Min. Peak δ	22.0	deg
Min. Peak c_a	0.0	psf

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Determine: Static factor of safety for cover system based on additional seismic loads.

Procedure:

Determine the static stability of the veneer cover system to determine the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters. Account for seismic loads (C_s) as noted in Figure 6.

Assumptions:

In addition to the static case assumptions:
1. Seismic force acts on the centroid of the cover soil.
2. Seismic force is horizontal.
3. Deformation analysis not required.

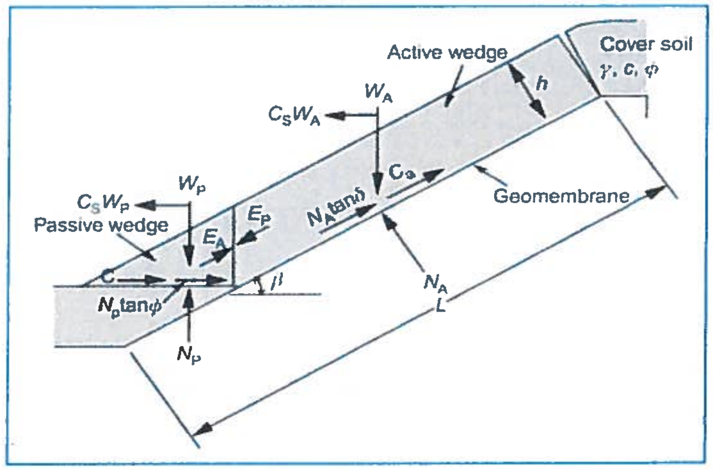

Figure 6. Limit equilibrium forces involved in pseudostatic analysis using average seismic coefficient.

Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	90.0	110.0	105.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	130.7	pcf	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	18.43	degrees	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	110.00	ft	Design - see Sheet 10
Lng. of Slp. Meas. Along Geom. (L)	-	-	347.94	ft	
Inter. Frict. Angle for DL & Geom. (δ)	-	-	31.6	deg	Minimum values to achieve Safety Factor
Adhesion for DL & Geom. (c_a)	-	-	0.0	psf	
Avg. Seismic Coeffcient (C_S or K_S)	-	-	**0.266**	%g	**Seismic Coefficient Calcs**
Required Factor of Safety (FS_R)	-	-	**1.00**		**Min. req'd FS for seismic conditions**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right)$$ = **89,228 lbs/ft**

$$N_A = W_A \cos\beta$$ = **84,652 lbs/ft**

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right)$$ = **- lbs/ft**

$$E_A = \frac{(FS)(C_S W_A + N_A \sin\beta)}{(FS)\cos\beta} - \frac{(N_A \tan\delta + C_a)\cos\beta}{(FS)\cos\beta}$$ = **1,149 lbs/ft**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta}$$ = **872 lbs/ft**

$$C = \frac{ch}{\sin\ \beta}$$ = **633 lbs/ft**

$$E_P = \frac{C + W_P \tan\phi - C_S W_P (FS)}{\cos\beta (FS) - \sin\beta \tan\phi}$$ = **1,007 lbs/ft**

$$N_P = W_P + E_P \sin\beta$$ = **1,190 lbs/ft**

Seismic Factor of Safety (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (C_S W_A + N_A \sin\beta)\cos\beta + C_S W_P \cos\beta$$ = **48,127 lbs/ft**

$$b = -\begin{bmatrix}(C_S W_A + N_A \sin\beta)\sin\beta\tan\phi \\ + (N_A \tan\delta + C_a)\cos^2\beta \\ + (C + W_P \tan\phi)\cos\beta\end{bmatrix}$$ = **(55,303) lbs/ft**

$$c = (N_A \tan\delta + C_a)\cos\beta \sin\beta \tan\phi$$ = **7284 lbs/ft**

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$ = **1.00**

Allow Exceeds Req'd - OK		
FS_R =	1.00	
FS_A =	1.00	
Min. Peak δ	31.6	deg
Min. Peak c_a	0.0	psf

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Determine: Static factor of safety for cover system based on residual strength.

Procedure:

Determine the static stability of the veneer cover system to evaluate the minimum required interface friction angle for all engineered components of the cover system. Balance the forces as shown in Figure 1 and the required factor of safety (FS) then solve for minimum interface shear strength parameters.

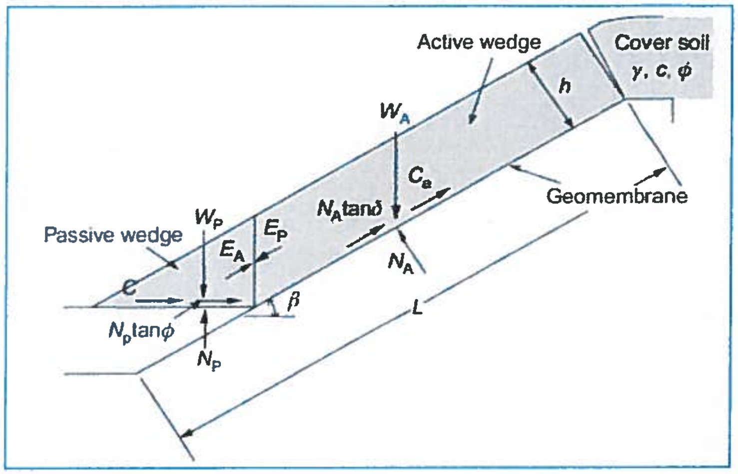

Figure 7. Limit equilibrium forces involved in a finite length slope analysis for a uniformly thick cover soil

Assumptions:

No additional assumptions

Veg. Cover Soil (VC)/Prot. Cover Soil (CS) and Slope Parameters

	VC	CS	Utilized		Source
Thickness (h)	0.5	1.5	2.0	ft	Design - see Sheet 18
Dry Unit Weight (γ_D)	90.0	110.0	105.0	pcf	Assumed values based on lab testing
Mois. Cont. (field cond.) (w_F)	26.0	24.0	24.5	%	Assumed values based on lab testing
Avg Field Unit Wt (γ)	-	-	130.7	pcf	
Reference Stress	-	-	**248.0**	psf	
Min. Friction Angle (ϕ)	25.0	25.0	25.0	deg	Assumed (conservative for clay)
Min. Cohesion (c)	100.0	100.0	100.0	psf	Assumed (conservative for clay)
Slope Angle Beneath the Geom. (β)	-	-	18.43	deg	Design - see Sheet 10
Ht. of Slp. Meas. Along Geom. (H_L)	-	-	110.00	ft	
Lng. of Slp. Meas. Along Geom. (L)	-	-	347.94	ft	Design - see Sheet 10
Inter. Frict. Angle for DL & Geom. (δ)	-	-	**21.0**	deg	**Minimum values to achieve Safety Factor**
Adhesion for DL & Geom. (c_a)	-	-	**0.0**	psf	
Required Factor of Safety (FS_R)	-	-	**1.20**		**Min. req'd FS**

Active Wedge Calculations

Determine the total weight of the active wedge (W_A),the effective force normal to the failure plan of the active wedge (N_A), the adhesive force between the cover soil of the active wedge and the geomembrane (C_a), and the interwedge force acting on the active wedge from the passive wedge (E_A) using the following eqs:

$$W_A = \gamma h^2\left(\frac{L}{h} - \frac{1}{\sin\beta} - \frac{\tan\beta}{2}\right)$$ = **89,228 lbs/ft**

$$N_A = W_A \cos\beta$$ = **84,652 lbs/ft**

$$C_a = c_a\left(L - \frac{h}{\sin\beta}\right)$$ = **- lbs/ft**

$$E_A = \frac{(FS)(W_A - N_A\cos\beta) - (N_A\tan\delta + C_a)\sin\beta}{\sin\beta(FS)}$$ = **1,130 lbs/ft**

Passive Wedge Calculations

Determine the total weight of the passive wedge (W_P),the effective force normal to the failure plan of the passive wedge (N_P), the cohesive force along the failure plane (C), and the interwedge force acting on the passive wedge from the active wedge (E_P) using the following eqs:

$$W_P = \frac{\gamma h^2}{\sin 2\beta} = 872 \text{ lbs/ft}$$

$$C = \frac{ch}{\sin \beta} = 633 \text{ lbs/ft}$$

$$E_P = \frac{C + W_P \tan\phi}{\cos\beta(FS) - \sin\beta\tan\phi} = 1{,}049 \text{ lbs/ft}$$

$$N_P = W_P + E_P \sin\beta = 1{,}203 \text{ lbs/ft}$$

Static Factor of Safety for Residual Strength (Solved for Iteratively)

Determine the calculated Factor of Safety (FS_A) using a quadratic equation relationship where the constants are defined as follows:

$$a = (W_A - N_A \cos\beta)\cos\beta = 8{,}461 \text{ lbs/ft}$$

$$b = -\left[\begin{array}{l}(W_A - N_A\cos\beta)\sin\beta\tan\phi \\ +(N_A\tan\delta + C_a)\sin\beta\cos\beta \\ +\sin\beta(C + W_P\tan\phi)\end{array}\right] = (11{,}389) \text{ lbs/ft}$$

$$c = (N_A\tan\delta + C_a)\sin^2\beta\tan\phi = 1{,}514 \text{ lbs/ft}$$

$$FS_A = \frac{-b + \sqrt{b^2 - 4ac}}{2a} = 1.20$$

Results Have Converged		
FS_R =	1.20	
FS_A =	1.20	
Min. Resid. δ	21.0	deg
Min. Resid. c_a	0.0	psf

Attachment B

Minimum Shear Strength Envelope

MINIMUM REQUIRED SHEAR STRENGTH FOR BOTTOM LINER VENEER STABILITY

The following chart depicts the minimum required shear strength values along the interfaces of the engineered components of the bottom liner system under relatively light uniform loads up to the reference load as illustrated in **Figure A.1**. The envelope depicts the peak shear strength used in the veneer stability analyses represented in the form of the minimum friction angle (ϕ) required to maintain the required factors of safety in the veneer stability analysis. All engineered components of the bottom liner system located adjacent to an interface along the slope should have a combination of shear strength parameters (i.e. cohesion/adhesion and friction angle, ϕ) where the mininmum strength for a given normal stress exceeds that of the peak shear strength envelope depicted in **Figure A.1**. Because the normal stress in the veneer analysis is due to a uniform load from the overlying material, a reference stress representing the commensurate normal stress applied from those materials is included on the chart for use in determining appropriate confining pressure for laboratory testing.

Using this reference stress, the acceptable combination of cohesion/adhesion and friction angle are plotted on **Figure A.2**. Any combination plotting above and to the right of the peak value curve represents acceptable values for this application as long as the field loads are equal to or less than the reference stress.

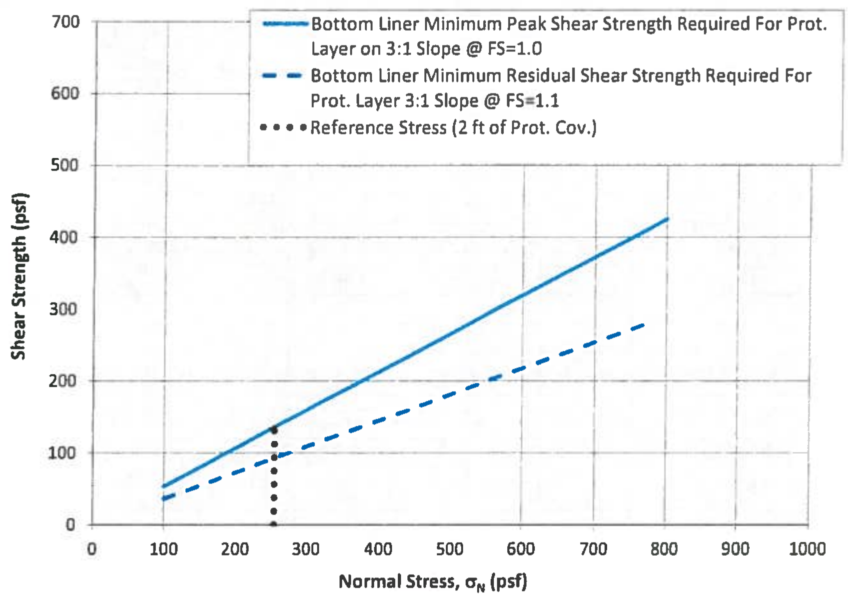

FIGURE A.1

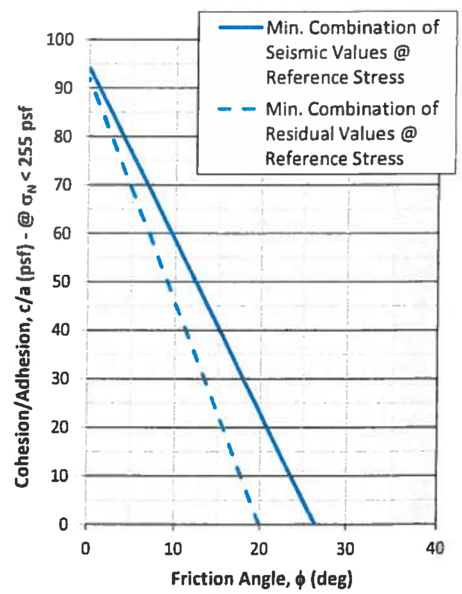

Figure 2: Bottom Liner Zone of Acceptable Values Curves

MINIMUM REQUIRED SHEAR STRENGTH FOR GLOBAL STABILITY

The following chart depicts the minimum required shear strength values along the interfaces of the engineered components of the liner system under relatively light uniform loads up to the reference load as illustrated in **Figure B.1**. The envelope depicts the peak shear strength used in the veneer stability analyses represented in the form of the minimum friction angle (ϕ) required to maintain the required factors of safety in the stability analysis. All engineered components of the final cover system located adjacent to an interface along the slope should have a combination of shear strength parameters (i.e. cohesion/adhesion and friction angle, ϕ) where the mininmum strength for a given normal stress exceeds that of the peak shear strength envelope depicted in **Figure B.1**. Because the normal stress in the veneer analysis is due to a uniform load from the overlying material, a reference stress representing the commensurate normal stress applied from those materials is included on the chart for use in determining appropriate confining pressure for laboratory testing.

Using this reference stress, the acceptable combination of cohesion/adhesion and friction angle are plotted on **Figure B.2**. Any combination plotting above and to the right of the peak value curve represents acceptable values for this application as long as the field loads are equal to or less than the reference stress.

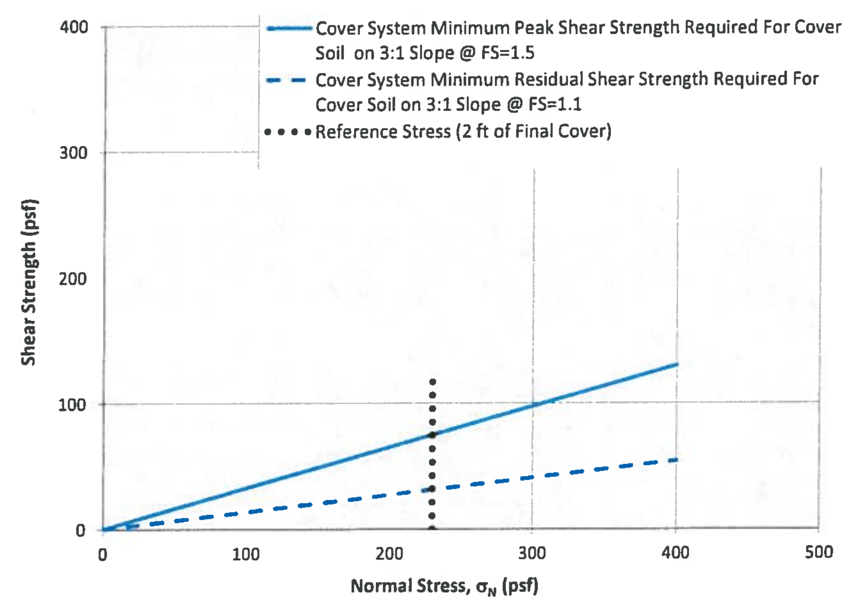

FIGURE B.1

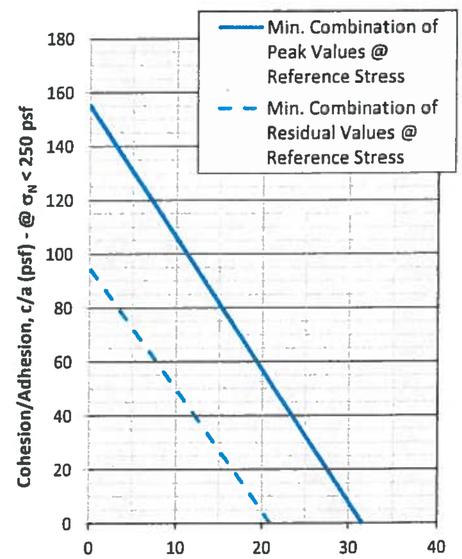

Figure 1: Cover Zone of Acceptable Values Curve

APPENDIX F.4B

Liquefaction Analysis (Rev 3)

AECOM

AECOM

Job	**Scepter Waverly Landfill-East Phase**	Project No.	**60398526**	Sheet	**1 Of 10**
Description	**Liquefaction Analysis (Rev 3)**	Computed by	**MW**	Date	**04/26/17**
		Checked by	**NG**	Date	**05/3/17**

I. PURPOSE

The purpose of this analysis is to evaluate the potential for liquefaction of the foundations soils for the proposed Class II Disposal Facility at Scepter in Waverly, Tennessee. The Tennessee Department of Environment and Conservation (TDEC) Solid Waste Processing and Disposal Regulations do not provide prescriptive standards related to liquefaction analysis. These calculations are being provided pursuant to rule 400-11-01-.04(2)(a)3 which states the facility must be located, designed, constructed, and maintained, and closed in such a manner as to minimize to the extent practicable the potential for releases of solid waste, solid waste constituents, or other potentially harmful material to the environment except in a manner authorized by state law.

II. SITE AND PROJECT DESCRIPTION

The liquefaction analysis was performed as part of the industrial waste permit application for the Scepter Site located in Waverly, Tennessee.

The proposed site will be permitted as a new industrial solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management.

The following sections summarize the methodology, assumptions, and results of the liquefaction analysis in the facility area. For further detail on the specific calculations performed, refer to the corresponding data provided in the **Attachments**.

III. SUMMARY OF SUBSURFACE CONDITIONS

In support of the liquefaction analysis, a hydrogeologic exploration program was performed recently consisting of 7 borings and 2 monitoring wells within and around the proposed footprint of the facility and constituting approximately 494 feet of drilling. In addition, previously developed subsurface data within and in the vicinity of the proposed landfill footprint including over 15 additional borings and laboratory test data were utilized in the assessment. Detailed observations, soil sample test results, analyses, and conclusions concerning the subsurface conditions are presented in the Hydrogeologic Evaluation report for the permit. Additional information was obtained from a subsurface exploration performed by GA Technical Services in 1989. The following summarizes the in-situ conditions related to the liquefaction analysis.

a. Soil and Bedrock

The subsurface soil material generally consists residuum consisting of low to medium plasticity clay (CL), clayey silt (ML), clayey sand (SC), and silty sand (SM) based on the Unified Soil Classification System (USCS). Laboratory tested samples obtained from the site typically had between 27 to 73 percent of material by weight passing the #200 sieve with 20% or more finer than 0.005 mm. The plasticity index of the soils generally ranged between 5 and 15 percent indicating a cohesive material of low to medium plasticity. It should be stated that significant quantities of chert gravel were present in the soil matrix of the majority of samples tested. When corrected for gravel content, the effective percent fines of the samples tended to increase by 10% to 40%. As a result, it is expected that the majority of the overburden will behave as a cohesive material regardless of USCS classification. Natural moisture typically varied between 6 and 41 percent. Hydraulic conductivity test results varied from 4.7×10^{-7} cm/s to 7.5×10^{-7} cm/s for samples obtained from relatively undisturbed Shelby tubes.

Generally, weathered bedrock immediately underlies the unconsolidated soil material at the site. Sufficient data for bedrock elevations does not currently exist for the site, therefore, the depth to bedrock across the footprint is assumed to vary from 110 to 130 feet below ground surface (bgs). The uppermost bedrock is comprised primarily of the Fort Payne formation overlying the Chattanooga shale. These upper units are generally described as follows:

- The Fort Payne formation in the area constitutes the upper 100 to 200 feet of the ridges. This formation has weathered almost completely to a residuum of a chert silt/clay matrix and colluvium.

- The Chattanooga Shale is black, fissile, pyritic, carbonaceous often called bituminous shale which acts as a confining unit. The Chattanooga shale does not outcrop at the site and was encountered during drilling for the Scepter facility supply well at a depth of approximately 130 feet below ground surface (bgs).

b. Groundwater

Groundwater generally occurs in two zones at the site: shallow systems in the alluvial deposits beneath the area at depths ranging from approximately 35 to 80 feet bgs and the deep regional system within the carbonate aquifer in the Fort Payne formation.

The shallow water is semi-confined and its connection to the surface (i.e., recharge by surface infiltration) is evidenced by temperature and water level fluctuations in wells shortly following precipitation events. Local groundwater flow typically mimics the topography, and the flow direction is generally toward area drainage features. A topographically low area is present north of the Scepter landfill.

No ponds, springs, sinkholes, ephemeral seeps or wetlands were identified during site reconnaissance. A perennial water feature exists on the north side of the CSX railroad tracks which is identified as a blue line stream within Leach Hollow. This is the receiving stream for all site drainage which flows west to the Tennessee River.

The regional system in the vicinity of the sites is located within the carbonate aquifer. Groundwater within the regional aquifer is encountered in the Fort Payne formation. Recharge to the groundwater occurs primarily from infiltration of precipitation. The recharge waters ultimately migrates downgradient toward the Tennessee River. The Tennessee River flows south to north adjacent to the Scepter site. It is dammed to form Kentucky Lake near Lake City, Kentucky.

IV. LIQUEFACTION POTENTIAL

Liquefaction is defined as the sudden loss of soil strength due to the increase of pore water pressure during dynamic loading. The increased pore water pressure is due to volumetric strains caused by cyclic stresses commonly associated with earthquake shaking. Liquefaction occurs primarily in clean sands, non-plastic silty sands, non-plastic silt, and gravels. It has also been observed in sensitive clays non-plastic silts.

The soil matrix at the site is comprised of a base of fine-grained materials with varying amounts sand and gravel depending on depth. Some intervals have enough coarse grained particles to be considered a sand or a gravel and are therefore susceptible to liquefaction. To be comprehensive in this analysis, all depths below the proposed cut elevations will be evaluated for liquefaction potential. The potential for liquefaction will be quantified in the following steps described below.

a. Preliminary Screening

A preliminary screening procedure to determine if the in-situ soil has characteristics susceptible to liquefaction is usually recommended to determine if a more rigorous and in-depth analysis is necessary. A commonly used screening procedure is included in the USEPA document *RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Facilities*. This method suggests performing a more in-depth analysis if the soil material meets three or more of the following five criteria each of which are considered in detail below:

1. ***Geologic Age and Origin.*** If a soil layer is a fluvial (river), lacustrine (lake), or aeolian (wind) deposit of Holocene age, a greater potential for liquefaction exists than for till, residual deposits, or older deposits.

Published geologic data (Fullerton, David S., Bush, Charles A., and Pennell, Jean N., 2003, "Map of Surficial Deposits and Materials in the Eastern and Central United States (East of 102° West Longitude)," United States Department of the Interior, United States Geological Survey) generally depicts the unconsolidated material that overlies the bedrock foundation in the central and eastern United States and includes the age, processes of origin, and aerial distribution of surficial geologic deposits and other materials that accumulated or formed during the past two million years. Based on the figure, unconsolidated materials in the region are generally nonstratified residual material derived by in-place solutioning of phosphatic carbonate rock, gypsum, or carbonate/gypsum-cemented rock, with no appreciable subsequent lateral transport by ice, water, wind, or gravity. Deposits are of Quaternary and Tertiary age.

Subsurface exploration indicates that the material encountered at the proposed project site consists predominantly of residuum consisting of low to medium plasticity clays with high gravel contents. However, as discussed, portions of the site contain fluvial deposits.

Based on the discussion above, the site soils partially match this criterion.

2. ***Fines Content and Plasticity.*** Liquefaction potential in a soil layer increases with decreasing fines content and plasticity of the soil. Soils having less than 15 percent (by weight) finer than 0.005 mm, a liquid limit less than 35 percent, and an in situ water content greater than 0.9 times the liquid limit may be susceptible to liquefaction (Seed and Idriss, 1982).

Seed et al. (1983) stated that based on both laboratory testing and field performance, the great majority of cohesive soils will not liquefy during earthquakes. Using these criteria originally stated by Seed and Idriss (above) and subsequently confirmed by Youd and Gilstrap (1999), in order for a cohesive soil to liquefy, it must meet all three criteria. If the cohesive soil does not meet all three criteria, then it is generally considered to not be susceptible to liquefaction (Day, 2002).

The results of the natural moisture content, sieve, and Atterberg limit tests performed on cohesive samples during the hydrogeologic exploration were reviewed. As part of the laboratory testing, ten samples were either classified as lean clay (CL) or clayey silt (ML), each with trace quantities of gravel (2.7% to 13.7%). Liquid limits of plastic materials ranged from 26 to 40 with an average value of 31. The natural moisture content ranged from 11.9 to 39% with an average value of 20.5% for all sampled materials. In addition, the average percent finer than the 0.005mm sieve (or clay sized particles) varied from 8% to 33% and averaged approximately. 26%. The ratio of natural moisture content to liquid limit (ω/LL) varied from 0.37 to 0.83. These data are summarized in **Table 1** below.

Table 1. Fines and Plasticity Summary

Soil Layer	Layer Elevation (ft msl)	Classification	% Clay		Liquid Limit, LL (%)	Natural Moisture Content, ω (%)	ω/LL	% Finer than 0.005mm
Lean Clay/Clayey Silt	Variable	CL/ML	8.2-32.3	(Average)	31	20.5	0.66	26
				(Minimum)	26	11.9	0.37	8
				(Maximum)	40	33	0.83	33

Based on the discussion above, on average, the cohesive materials do meet the criterion for liquefaction. However, individual test results did meet 2 of the 3 criteria for liquefaction of cohesive soil. Therefore, it will be conservatively assumed that the cohesive soils are susceptible to liquefaction. The results are summarized in **Table 2**.

Table 2. Summary of the Criteria (Silty Clay/Clayey Silt)

Criteria	Does Not Meet	Meets
Soil grain size percent finer than 0.005mm < 15%		✓
Soil liquid limit < 35		✓
Soil water content > 90% of liquid limit	✓	

In addition, the granular residual soils classified as silty sand (SM) or clayey sand (SC) with variable quantities of gravel are by definition susceptible to liquefaction. The liquid limit of these silty sands ranges from 25 to 40, the natural moisture content ranged from 6.1% to 39.8%, and the percentage of fine sized particles varied from 27.2% to 48.9%. As discussed, these materials demonstrate significant gravel content (2.3% to 39.7%, with an average of 20%). Gravel is inert with regard to liquefaction based on the porosity and lack of potential for pore water pressure increase. As a result, granular soils with high gravel content are less susceptible to liquefaction and the fines content of the material may be "amplified" in the behavior of the material under dynamic loading.

In any case, in regard to fines content, plasticity, and particle size distribution, it will be conservatively assumed that all site materials are susceptible to liquefaction.

3. ***Saturation.*** Although low water content soils have been reported to liquefy, at least 80 to 85 percent saturation is generally deemed to be a necessary condition for soils liquefaction.

Based on the results of the hydro-geologic exploration, piezometer installation, piezometer modeling, and potentiometric data analysis, the groundwater surface is present at a depth 35 to 80 feet below the existing ground surface at the site. Following mass grading to establish the subgrade elevation within the limits of the proposed landfill footprint, the potentiometric surface will be approximately 10 feet below the top of the the protective cover and within the proposed geologic buffer. Therefore, it can be conservatively assumed for the foundation soils to be at 85% saturated at all times.

Based on the above discussion, site soils do match this criterion.

4. ***Depth Below Ground Surface.*** Liquefaction is generally not likely to occur in soil layers entirely more than 50 feet below the ground surface.

Bedrock is 100+ feet below existing grades at the site. Therefore, bedrock surface will not limit the range of potentially liquefiable soils.

Potentially liquefiable soils exist at various depths across the site including depths from proposed landfill cut elevations to 50 feet below those cut elevations. Therefore, this material does match this criterion.

It should also be noted that as the landfill is constructed, waste material will accumulate and provide additional overburden pressure and confinement of the foundation materials as the landfill develops to the extent that the majority of the site will eventually have more than 50 feet of confining material.

5. ***Soil Penetration Resistance.*** Seed et al, 1985, state that soil layers with a normalized SPT blow count less than 22 have been known to liquefy. Marcuson et al, (1990), suggest an SPT value of less than 30 as the threshold to use for suspecting liquefaction potential.

Raw SPT N-values (uncorrected) obtained during the subsurface exploration were very high. These high blow counts recorded at most locations and depths across the site can partly be attributed to the high amount of residual chert fragments found in the soil matrix. Where the SPT N-value met or exceeded 50 blows per foot, an input N-value of 50 was utilized as a degree of conservatism. In addition, SPT N-values judged to be elevated by the presence of gravel in the soil matrix were corrected in general accordance with ASTM D 6066. Individual SPT-N values were only corrected if the initial or resulting N-value was decreased to less than 50 blows per foot. The results of the N-value correction are summarized below in **Table 3.**

Table 3. Summary of N-values corrected for gravel content

Boring	Depth Interval (feet)	Initial N-value (bpf)	Corrected N-value (bpf)
B-4	13.5-15	48	36
B-4	18.5-20	55	43
MW-4	18.5-20	26	16
MW-4	23.5-25	21	19
MW-E2	3.5-5	61	35
PZ-2	8.5-10	43	28
PZ-3	58.5-60	23	16
PZ-6	8.5-10	31	22
PZ-6	13.5-15	27	20
PZ-6	18.5-20	43	16
PZ-6	23.5-25	38	26
1	3.5-5	50	38
6	8.5-10	29	22

The SPT N-values were also corrected for overburden, energy, borehole diameter, rod length and sampling method using the procedure described in Youd and Idriss (2001). After the N-values were normalized and corrected to an equivalent clean sand value based on the influence of the fines content of the soils ($(N_1)_{60\text{-}cs}$), the majority (the statistical mode) of the values were greater than 50.

The lowest corrected N-values ($(N_1)_{60\text{-}cs}$) recorded from the subsurface exploration were 11, 24, and 34. These represent outlying values compared to the total quantity of SPT tests performed.

Based on the discussion above, the site soils do not match this criterion.

b. Results of Preliminary Screening

The results of the preliminary screening criteria are summarized in **Table 4** as follows:

Table 4. Liquefaction Screening Criteria Summary

	Criteria	Doesn't Meet	Partially Meets	Meets
1	Geologic Age and Origin		✓	
2	Fines Content and Plasticity*			✓
3	Saturation			✓
4	Depth Below Ground Surface			✓
5	Soil Penetration Resistance	✓		

V. PRELIMINARY SCREENING ANALYSIS CONCLUSIONS

Overall, although not all the criteria necessary for liquefaction triggering to occur were met in the preliminary screening analysis, it was determined that the liquefaction triggering analysis should be performed in any event. The triggering analysis methodology and results are described in the following sections.

VI. LIQUEFACTION ANALYSIS METHODOLOGY

The liquefaction analysis was performed using the most current form of the procedure as described in Idriss and Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12. This method computes the factor of safety against liquefaction by computing the cyclic stress ratio and cyclic resistance ratio. Specifically, this procedure consider a stress-based approach to evaluate the potential for liquefaction triggering, and compare the earthquake-induced cyclic stress ratios (CSR) with the cyclic resistance ratios (CRR) of the soil in terms of a factor of safety as follows.

The CSR, at a given depth (z), is expressed as 65-percent of the ratio of maximum earthquake-induced cyclic shear stress (τ_{max}) to the vertical effective overburden stress (σ'_v). The maximum cyclic shear stress is estimated using the Seed-Idriss Simplified Liquefaction Procedure, which incorporates the Peak

Horizontal Ground Acceleration coefficient (PGA), total overburden stress (σ_v), and shear stress reduction factor that accounts for dynamic soil response (r_d). The value of CSR is customarily adjusted to a reference σ'_v = 1 atm for an earthquake with mean moment Magnitude (M) = 7.5.

The CRR is the cyclic stress ratio at which liquefaction occurs during an earthquake. It is obtained from case history-based semi-empirical correlations with SPT or $SCPT_u$ values recorded at sites with level ground conditions and $\sigma'_v \approx 1$ atm for an earthquake with M = 7.5. The CRR is a function of a soil's fines content (FC), relative density and effective stress, and penetration resistance. The CRR is also dependent on the duration of shaking, and is adjusted to the site-specific design earthquake using a Magnitude Scaling Factor (MSF). Other correction factors to adjust for confinement stress (K_σ) and sloping ground (K_α) may also be applied. The minimum acceptable factor of safety used in the analysis is 1.1.

As discussed, the liquefaction analysis was performed for all borings where standard penetration testing was performed. The results of the triggering analysis are contained in **Attachment A**. The procedure and results are summarized below along with the input variable and rationale.

VII. SELECTION OF PARAMETERS

a. Subsurface Profiles

Profiles for the liquefaction analyses were taken directly from the boring logs included in the hydrogeologic exploration and historical subsurface data gathered for the project. Specifically, the water levels were based on potentiometric analyses derived from monitoring the on-site piezometers and either the water levels measured in the boring at the time of drilling or conservatively assigned based on the potentiometric surface of the groundwater at the site.

b. Material Properties

Material properties pertinent to the procedure include the SPT N-value, soil unit weight above and below the water table, and fines content. SPT N-values were taken directly from the results of field testing, as given on the boring logs and corrected to $(N_1)_{60\text{-}cs}$ using the method described above.

Based on laboratory testing data results, the average moist unit weight was found to be 116 pcf above the water table and 121 pcf below the water table. The fines content of the material was interpolated from laboratory data or assumed based on published standards where laboratory data was not available. Based on the geotechnical testing results, no material contained less than 25% fines.

The effect of the landfill surcharge on the in-situ stresses was conservatively ignored in all analyses. The surcharge will increase confining stress and thus provide additional resistance against liquefaction.

In addition, some of the historical borings were terminated at elevations above the geologic buffer. Although not specifically pertinent to the design of this facility, a liquefaction triggering analysis has been performed on these borings in order to provide further data on the potential for liquefaction triggering within this seismic region and local setting.

Borings logs utilized to perform the liquefaction triggering analysis and laboratory test results used to assess the samples are provided in AECOM's Hydrogeologic Evaluation report.

c. Design Seismic Event

The horizontal seismic load value and design earthquake magnitude are based on seismic hazard mapping from the United States Geological Survey (USGS) and is further detailed in the Seismic Coefficient Calculations computed as part of the permit application. The seismic coefficient (a_{MAX}) corresponds to the peak ground acceleration with a 2% probability of exceedance in 50 years at the site. These values are summarized below in **Table 5.**

Table 5. Seismic Design Parameters for the Liquefaction Analysis

Parameter	Value
Seismic Coefficient, a_{MAX}	0.266g
Earthquake Magnitude, M_W	7.7

VIII. LIQUEFACTION ANALYSIS RESULTS

The quoted factor of safety represents the minimum computed value for any natural layer within an individual boring profile. At the vast majority of locations tested, the factor of safety against liquefaction triggering was greater than 2.0. In isolated locations, factors of safety between 1.2 and 2.0 were obtained prior to correcting for fines content (FC). The factors of safety are adjusted for a magnitude scaling factor (MSF) to adjust from the Simplified Method design earthquake magnitude of 7.5 to a site specific earthquake magnitude of 7.7. The results of the liquefaction triggering analysis where a factor of safety of less than 2 are summarized below.

Table 6. Liquefaction Analysis Summary of Results

				Before Correcting for FC		After Correcting for FC	
Boring	Elevation	Material Description	Cyclic Stress Ratio, CSR	Cyclic Resist. Ratio, $CRR_{7.5}$	Factor of Safety	Cylic Resist. Ratio, $CRR_{7.5\text{-}cs}$	Factor of Safety
PZ-3	468.5-470	CL	0.250	0.376	1.50	0.66	< 2
PZ-6	454.5-456	SM	0.293	0.410	1.40	0.798	< 2
MW-4	372.5-371	CL/ML	0.180	0.233	1.25	0.374	1.82
MW-4	367.5-366	CL/ML	0.190	0.248	1.29	0.423	1.84

For all other intervals tested, the factor of safety against liquefaction triggering was 2 or greater. Detailed calculations including a sample calculation are included in **Attachment A**.

IX. CONCLUSIONS

The lowest computed factor of safety at any boring location was 1.82 after correcting for fines. In addition, the vast majority of borings indicated a factor of safety of at least 2.0. The factor of safety will improve as the landfill is constructed due to the increase in overburden pressure and confinement. Further, conservative assumptions were made as to whether the site soils were potentially liquefiable. Based on these considerations and the overall results of the detailed liquefaction triggering analysis, liquefaction is not anticipated and no special engineering features are required with respect to liquefaction.

X. REFERENCES

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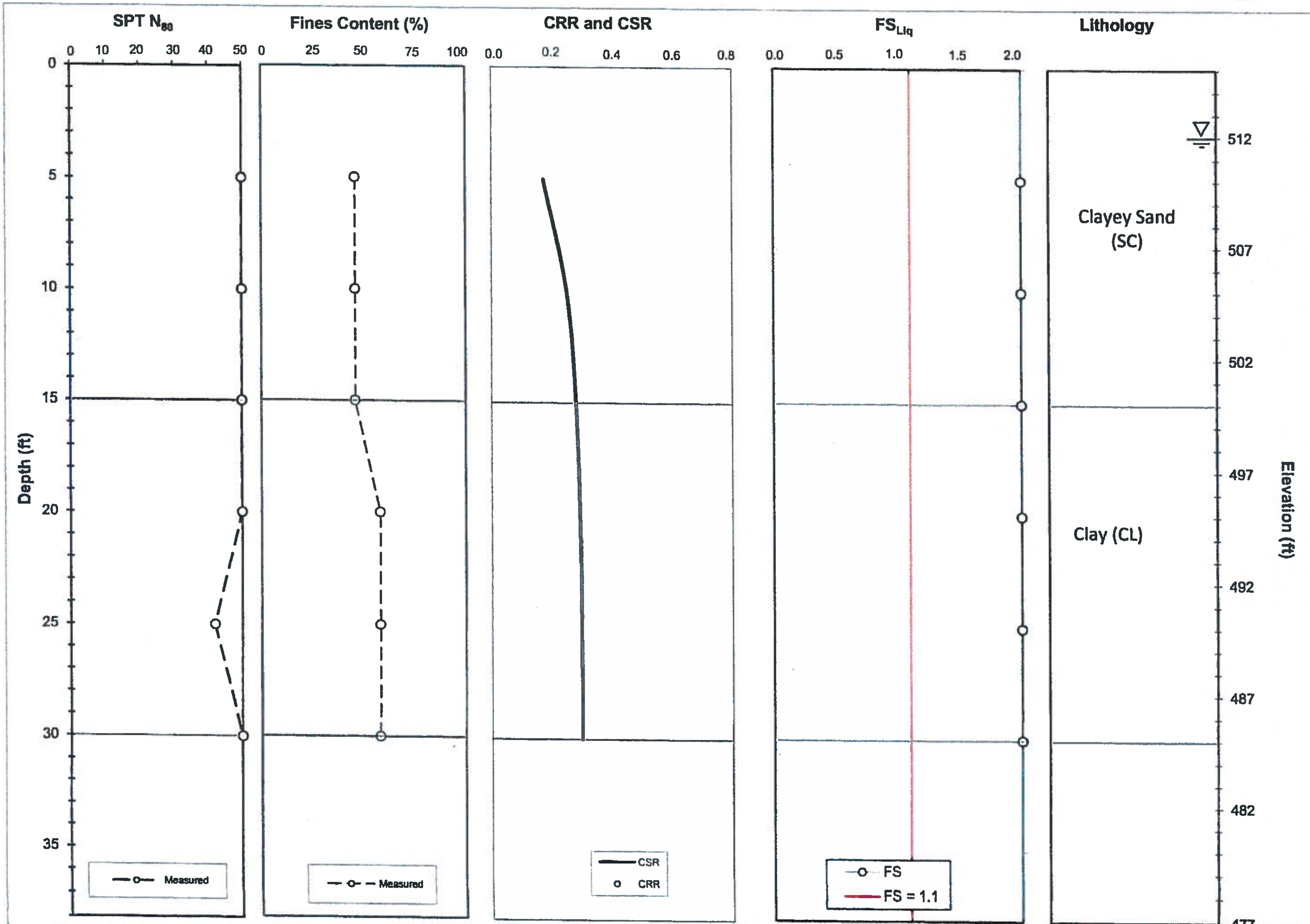
XI. ATTACHMENTS

Attachment A Liquefaction Triggering Analysis Output and Detailed Calculation Example

Attachment A

Liquefaction Triggering Analysis Output and Detailed Calculation Example

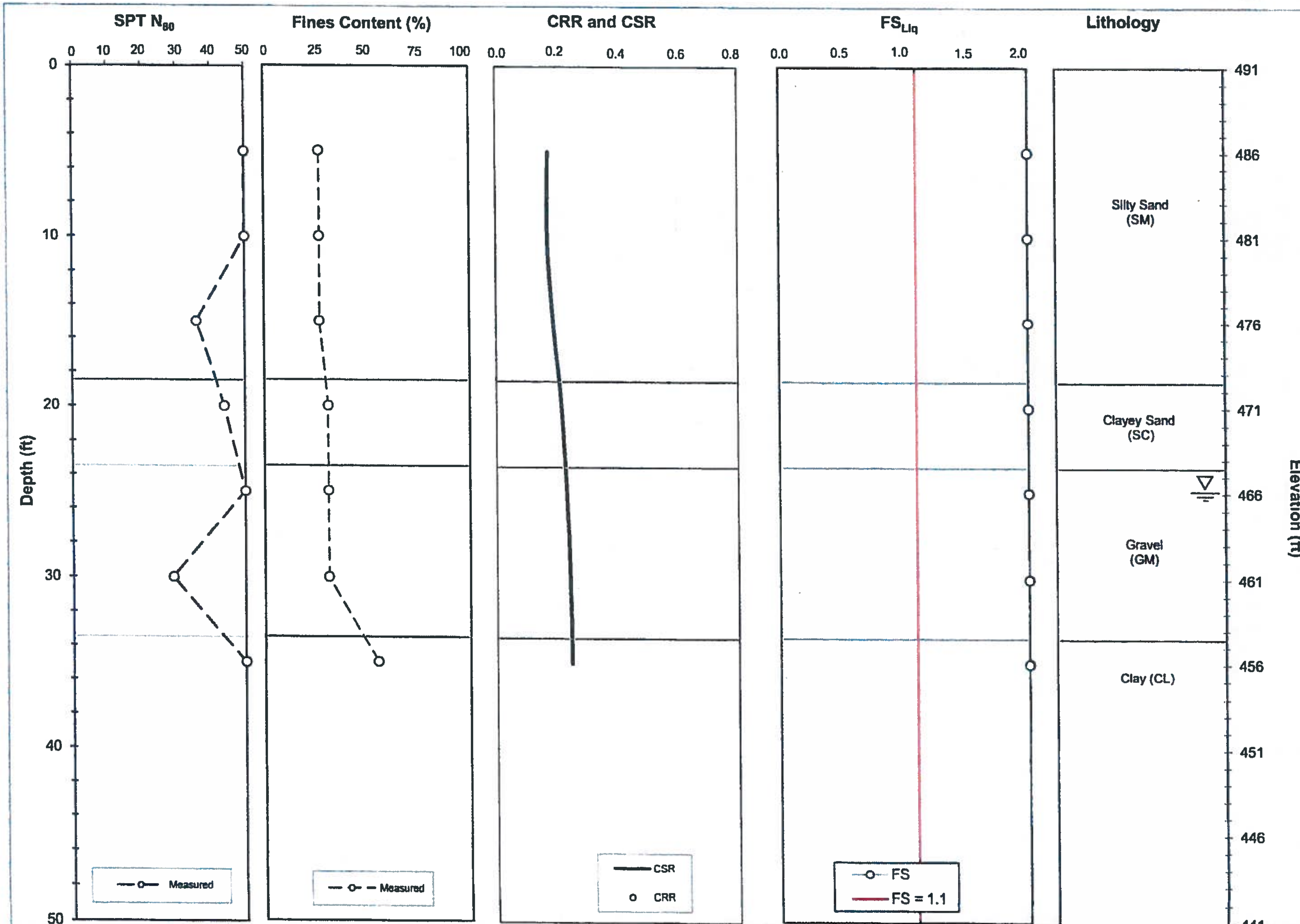


Peak ground acceleration, PGA = 0.266g
Earthquake Magnitude, M = 7.7
Ground Motion: 2,475-Year Modal

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Scepter Waverly Landfill: B-1	
Project: Scepter Landfill	Date: January 2017

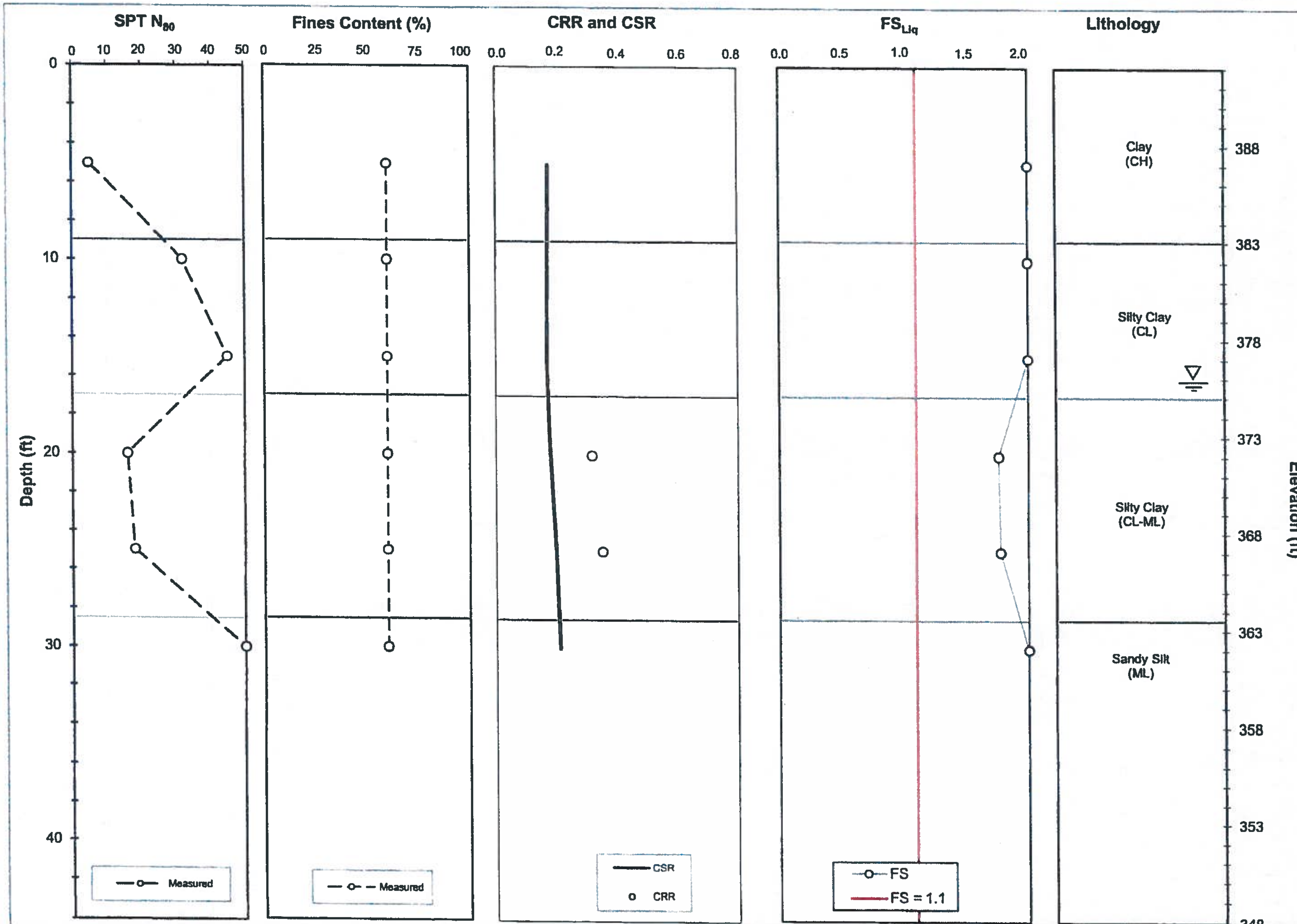
AECOM



Peak ground acceleration, PGA = 0.266g
Earthquake Magnitude, M = 7.7
Ground Motion: 2,475-Year Modal

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

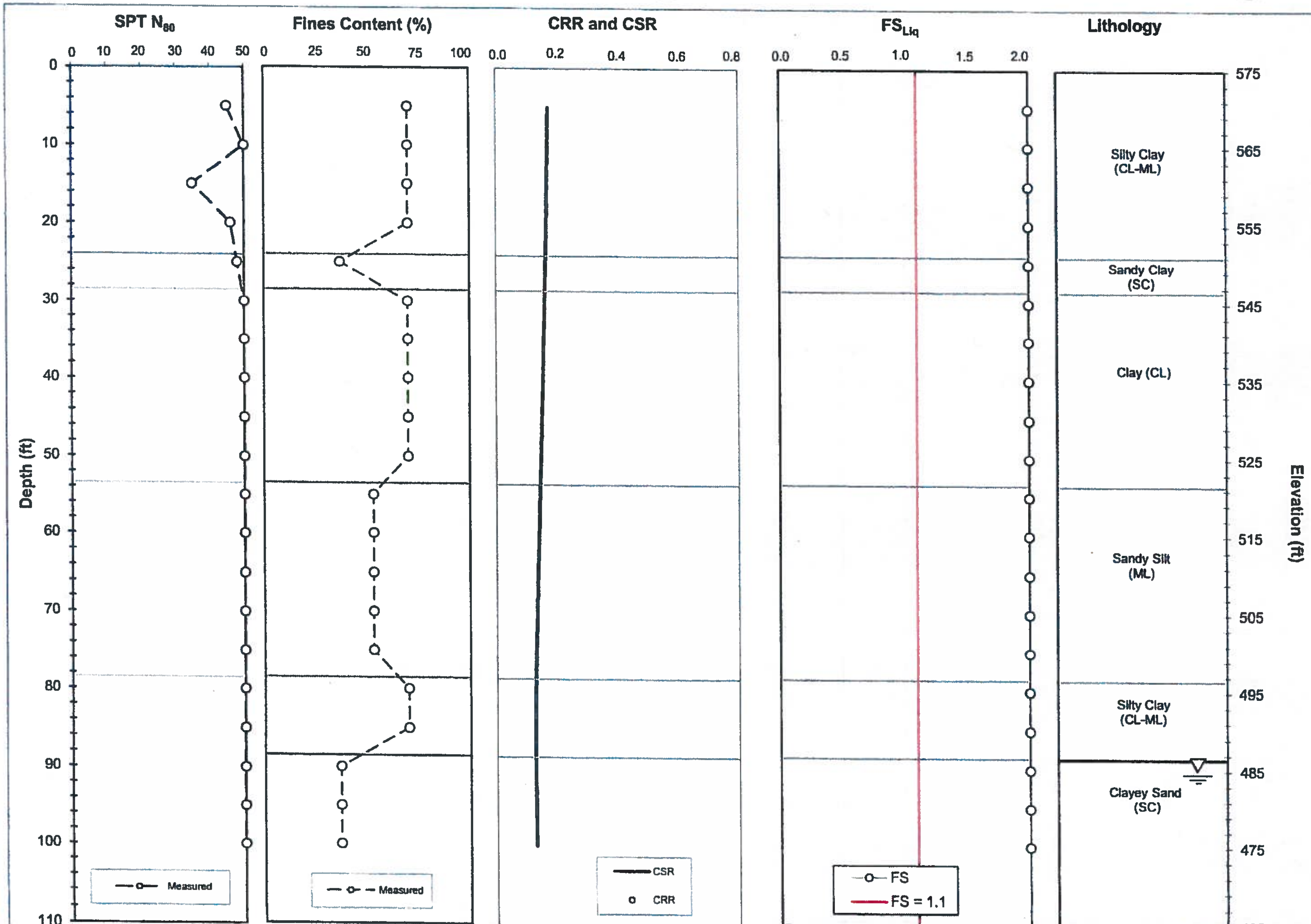
Title: Scepter Waverly Landfill: B-4		AECOM
Project: Scepter Landfill	Date: January 2017	



Peak ground acceleration, PGA = 0.266g
Earthquake Magnitude, M = 7.7
Ground Motion: 2,475-Year Modal

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

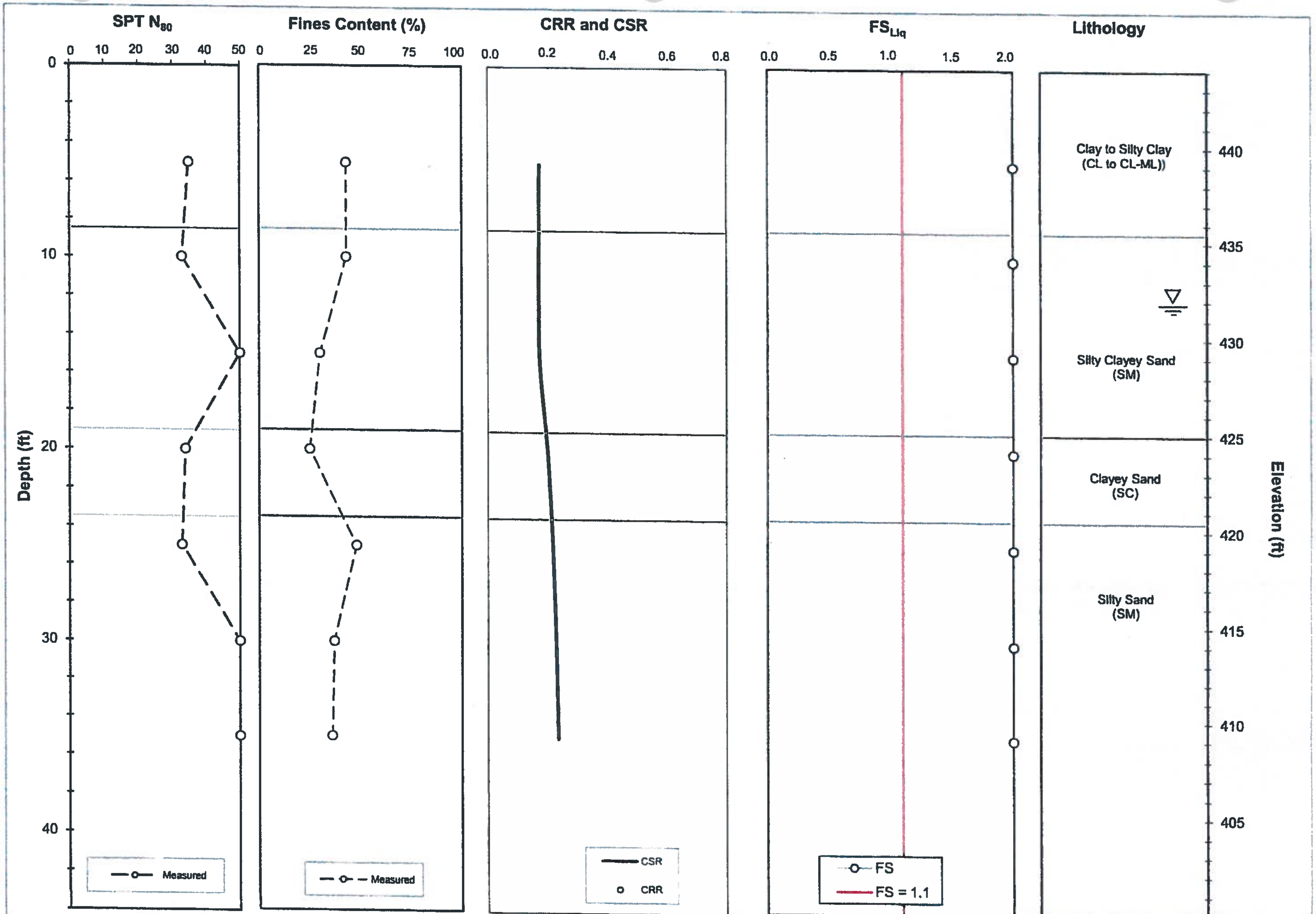
Title: Scepter Waverly: MW-4		AECOM
Project: Scepter Landfill	Date: January 2017	



Peak ground acceleration, PGA = 0.266g
Earthquake Magnitude, M = 7.7
Ground Motion: 2,475-Year Modal

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Scepter Waverly Landfill: MW-E1		AECOM
Project: Scepter Landfill	Date: January 2017	

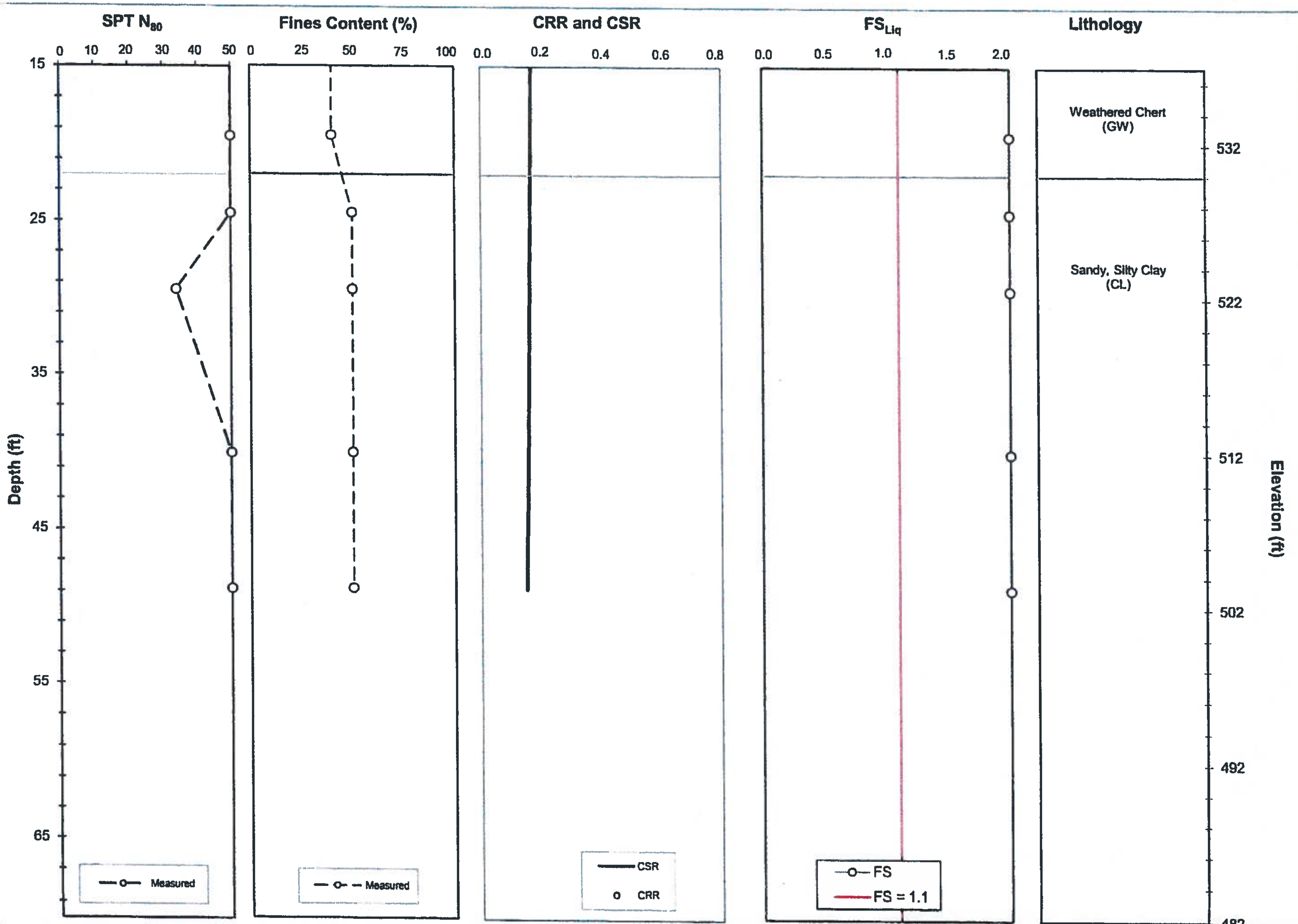


Peak ground acceleration, PGA = 0.266g
Earthquake Magnitude, M = 7.7
Ground Motion: 2,475-Year Modal

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Scepter Waverly Landfill: MW-E2	
Project: Scepter Landfill	Date: January 2017

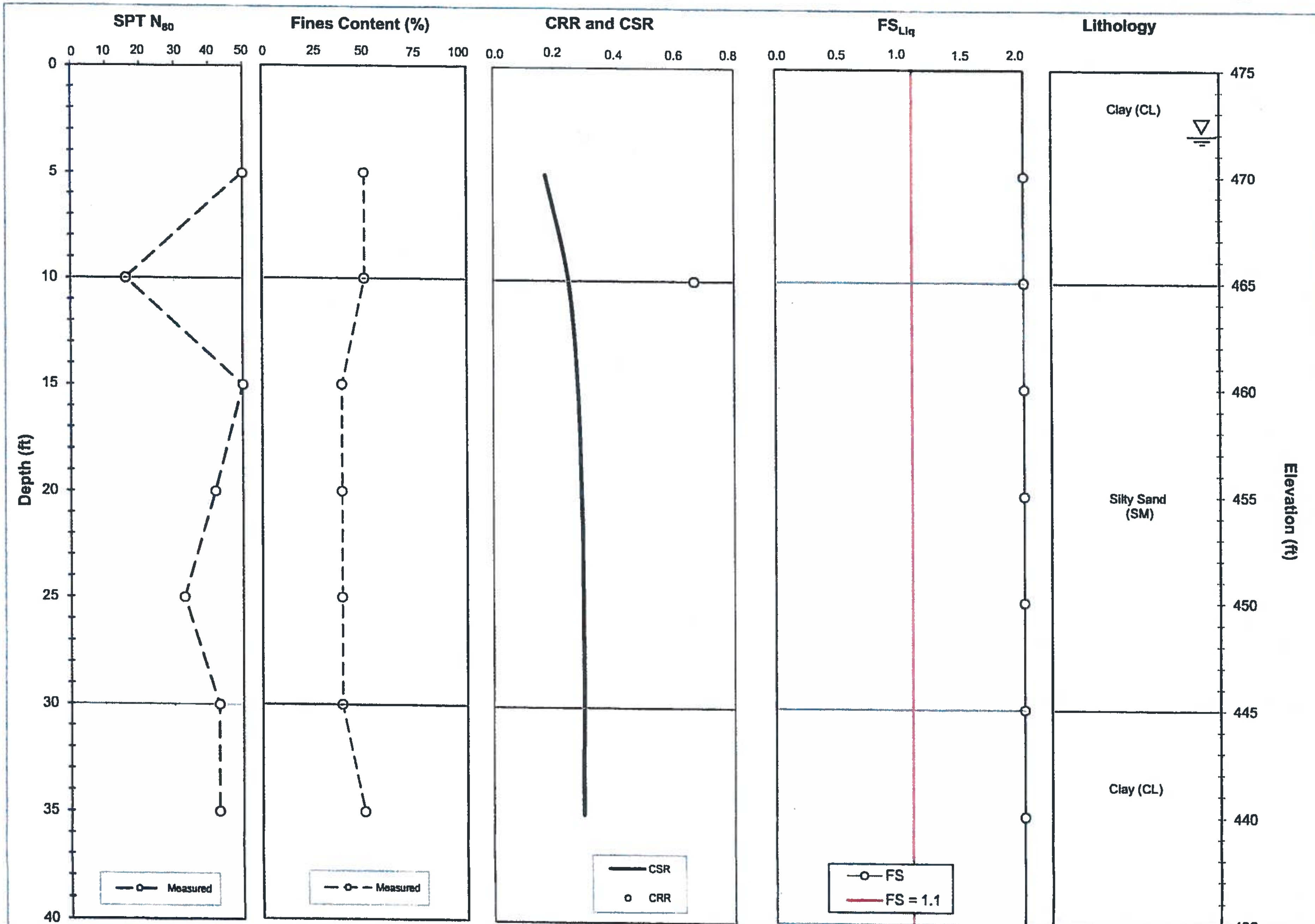
AECOM



Peak ground acceleration, PGA = 0.266g
Earthquake Magnitude, M = 7.7
Ground Motion: 2,475-Year Modal

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

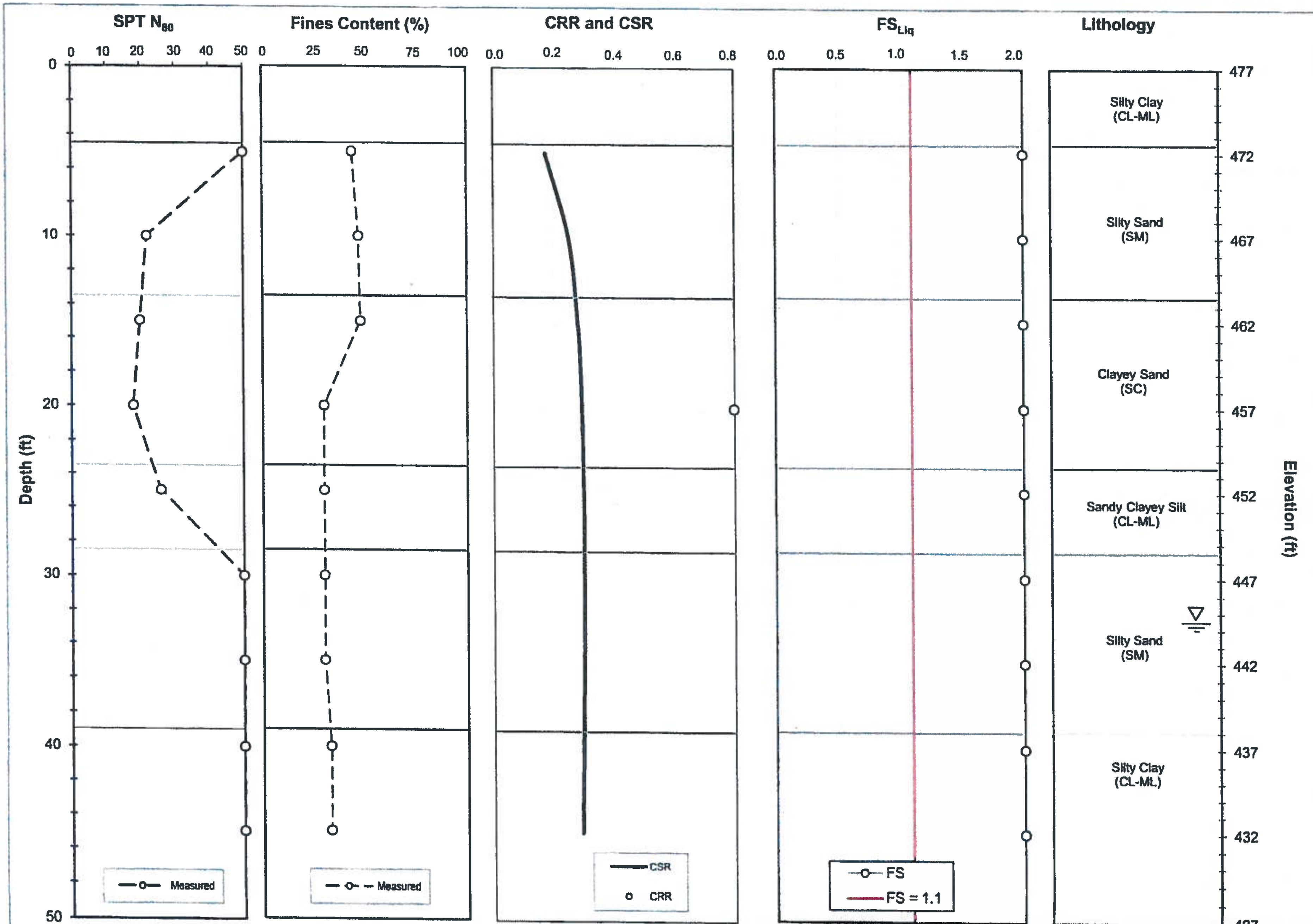
Title: Scepter Waverly Landfill: No. 5		AECOM
Project: Scepter Landfill	Date: January 2017	



Peak ground acceleration, PGA = 0.266g
Earthquake Magnitude, M = 7.7
Ground Motion: 2,475-Year Modal

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Scepter Waverly Landfill: PZ-3		AECOM
Project: Scepter Landfill	Date: January 2017	



Peak ground acceleration, PGA = 0.266g
Earthquake Magnitude, M = 7.7
Ground Motion: 2,475-Year Modal

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: Scepter Waverly: PZ-6		AECOM
Project: Scepter Landfill	Date: January 2017	

SPT-Based Liquefaction Triggering at Scepter, Inc. Waverly Landfill

Prepared for

Scepter Inc.

Scepter, Inc.
Waverly, Tennessee

Prepared by

AECOM

1000 Corporate Centre Drive, Suite 250
Franklin, Tennessee 37067-6209

Date: NOV-17-2016

Discussion:

The following calculation is an example of the SPT-based liquefaction triggering computed in Microsoft Excel. The liquefaction triggering is calculated for Scepter, Inc.'s Landfill in Waverly, Tennessee. The calculations are based on the following references:

1) Idriss and Boulanger, 2008, Soil Liquefaction during Earthquakes, EERI Publication MNO-12.
2) Idriss and Boulanger, 2010, SPT-Based Liquefaction Triggering Procedures, Report No. UCD/CGM-10-02.

Input Parameters

Peak ground acceleration $a_{max} := 0.266g$

Earthquake magnitude $M := 7.7$

Depth to water table at time of drilling $GWT_d := 3ft$

Depth to water table at time of earthquake $GWT_e := 3ft$

Average unit weight of soil above ground water table $\gamma_a := 116pcf$

Average unit weight of soil below ground water table $\gamma_b := 121pcf$

Unit weight of water $\gamma_w := 62.42pcf$

Borehole diameter $BH_d := 4.25in$

Rod stickup above ground at start of drive $H_{rs} := 5ft$

Boring total depth $d_{BH} := 78.5ft$

Ground surface elevation $GSE := 515ft$

Earthquake magnitude scaling factor for sand $MSF := \min\left[6.9 \cdot e^{\left(\frac{-M}{4}\right)} - 0.058, 1.8\right] = 0.95$

Sampler type

Standard sampler, set st = "standard"
With liner for dense sand and clay, set st = "liner, dense sand or clay"
With liner for loose sand, set st = "liner, loose sand"

sampler type, st := "standard"

Calculations for Boring No. B-1

Depth	Elevation	Measured Blow Count	Soil Type	Fines Content	Energy Ratio

$$D := \begin{pmatrix} 5 \\ 10 \\ 15 \\ 20 \\ 25 \\ 30 \end{pmatrix} ft \quad EL := GSE - D = \begin{pmatrix} 510.00 \\ 505.00 \\ 500.00 \\ 495.00 \\ 490.00 \\ 485.00 \end{pmatrix} ft \quad N_m := \begin{pmatrix} 50 \\ 50 \\ 50 \\ 50 \\ 42 \\ 50 \end{pmatrix} \quad USCS := \begin{pmatrix} \text{"SC"} \\ \text{"SC"} \\ \text{"SC"} \\ \text{"CL"} \\ \text{"CL"} \\ \text{"CL"} \end{pmatrix} \quad FC := \begin{pmatrix} 46 \\ 46 \\ 46 \\ 58 \\ 58 \\ 58 \end{pmatrix} \% \quad ER := \begin{pmatrix} 80 \\ 80 \\ 80 \\ 80 \\ 80 \\ 80 \end{pmatrix} \%$$

Depth to midpoint of each layer, $z := \begin{bmatrix} \frac{1}{2} D_1 \\ D_1 + \frac{1}{2}(D_2 - D_1) \\ D_2 + \frac{1}{2}(D_3 - D_2) \\ D_3 + \frac{1}{2}(D_4 - D_3) \\ D_4 + \frac{1}{2}(D_5 - D_4) \\ D_5 + \frac{1}{2}(D_6 - D_5) \end{bmatrix} = \begin{pmatrix} 2.50 \\ 7.50 \\ 12.50 \\ 17.50 \\ 22.50 \\ 27.50 \end{pmatrix} ft$

Measured value of delivered energy as a percentage of theoretical free-fall hammer energy, $C_E := \frac{ER}{60\%} = \begin{pmatrix} 1.33 \\ 1.33 \\ 1.33 \\ 1.33 \\ 1.33 \\ 1.33 \end{pmatrix}$

Correction factor for borehole diameter ($BH_d = 4.25 \cdot in$), $C_B := \left| \begin{array}{l} 1 \text{ if } BH_d \le 4.7in \\ 1.05 \text{ if } 4.7in < BH_d \le 6in \\ 1.15 \text{ otherwise} \end{array} \right. = 1.00$

Sampler correction factor, $C_S := \left| \begin{array}{l} 1 \text{ if } st = \text{"standard"} \\ 0.8 \text{ if } st = \text{"liner, dense sand or clay"} \\ 0.9 \text{ if } st = \text{"liner, loose sand"} \\ \text{"check sampler type" otherwise} \end{array} \right. = 1.00$

Correction factor for rod length, $i := 1..6$

$$C_{R_i} := \begin{cases} 0.75 & \text{if } H_{rs} + z_i < 3m \\ 0.80 & \text{if } 3m \le H_{rs} + z_i < 4m \\ 0.85 & \text{if } 4m \le H_{rs} + z_i < 6m \\ 0.95 & \text{if } 6m \le H_{rs} + z_i < 10m \\ 1 & \text{otherwise} \end{cases} \qquad C_R = \begin{pmatrix} 0.75 \\ 0.80 \\ 0.85 \\ 0.95 \\ 0.95 \\ 0.95 \end{pmatrix}$$

Standard penetration number corrected for field conditions,
$N_{60_i} := N_{m_i} \cdot C_{E_i} \cdot C_B \cdot C_S \cdot C_{R_i}$

$$N_{60} = \begin{pmatrix} 50.0 \\ 53.3 \\ 56.7 \\ 63.3 \\ 53.2 \\ 63.3 \end{pmatrix}$$

Total overburden stress, $\sigma_{vo_i} :=$
(at time of drilling)

$$\sigma_{vo_i} := \begin{cases} \gamma_a \cdot z_i & \text{if } z_i \le GWT_d \\ \gamma_a \cdot GWT_d + \gamma_b \cdot (z_i - GWT_d) & \text{otherwise} \end{cases} \qquad \sigma_{vo} = \begin{pmatrix} 13.89 \\ 42.73 \\ 71.70 \\ 100.67 \\ 129.64 \\ 158.60 \end{pmatrix} \cdot kPa$$

Effective overburden stress, $\sigma'_{vo_i} :=$
(at time of drilling)

$$\sigma'_{vo_i} := \begin{cases} \gamma_a \cdot z_i & \text{if } z_i \le GWT_d \\ \gamma_a \cdot GWT_d + (\gamma_b - \gamma_w) \cdot (z_i - GWT_d) & \text{otherwise} \end{cases} \qquad \sigma'_{vo} = \begin{pmatrix} 13.89 \\ 29.28 \\ 43.31 \\ 57.33 \\ 71.36 \\ 85.38 \end{pmatrix} \cdot kPa$$

Equivalent clean sand adjustment,

$$\Delta N := e^{\left[1.63 + \frac{9.7}{\frac{FC}{\%} + 0.01} - \left(\frac{15.7}{\frac{FC}{\%} + 0.01}\right)^2\right]} = \begin{pmatrix} 5.61 \\ 5.61 \\ 5.61 \\ 5.61 \\ 5.61 \\ 5.61 \end{pmatrix}$$

Equivalent clean sand adjustment for residual strength, $\Delta N_{rs_i} := \begin{cases} 5 & \text{if } FC_i > 75\% \\ 4 & \text{if } 75\% \geq FC_i > 50\% \\ 2 & \text{if } 50\% \geq FC_i > 25\% \\ 1 & \text{if } 25\% \geq FC_i > 10\% \\ 0 & \text{otherwise} \end{cases}$ $\quad \Delta N_{rs} = \begin{pmatrix} 2.00 \\ 2.00 \\ 2.00 \\ 4.00 \\ 4.00 \\ 4.00 \end{pmatrix}$

Initial overburden correction factor, $C_{Ni_i} := \min\left(\sqrt{\frac{1\text{atm}}{\sigma'_{vo_i}}}, 1.7\right)$ $\quad C_{Ni} = \begin{pmatrix} 1.70 \\ 1.70 \\ 1.53 \\ 1.33 \\ 1.19 \\ 1.09 \end{pmatrix}$

Initial corrected penetration resistance, $Ni_{1.60_i} := C_{Ni_i} \cdot N_{60_i}$ $\quad Ni_{1.60} = \begin{pmatrix} 85.00 \\ 90.67 \\ 86.68 \\ 84.20 \\ 63.39 \\ 68.99 \end{pmatrix}$

Iteration 1, $N_{1.60cs1} := Ni_{1.60} + \Delta N = \begin{pmatrix} 90.61 \\ 96.28 \\ 92.29 \\ 89.80 \\ 69.00 \\ 74.60 \end{pmatrix}$

Iteration 1, $C_{N1_i} := \min\left[\left(\frac{1\text{atm}}{\sigma'_{vo_i}}\right)^{0.784-0.0768\cdot\sqrt{\min\left(N_{1.60cs1_i}, 46\right)}}, 1.7\right]$ $\quad C_{N1} = \begin{pmatrix} 1.69 \\ 1.39 \\ 1.25 \\ 1.16 \\ 1.10 \\ 1.05 \end{pmatrix}$

Iteration 1, $N1_{1.60_i} := C_{N1_i} \cdot N_{60_i}$ $\quad N1_{1.60} = \begin{pmatrix} 84.35 \\ 73.93 \\ 70.87 \\ 73.57 \\ 58.34 \\ 66.25 \end{pmatrix}$

Iteration 2, $N_{1.60cs2} := N1_{1.60} + \Delta N = \begin{pmatrix} 89.96 \\ 79.54 \\ 76.48 \\ 79.18 \\ 63.95 \\ 71.86 \end{pmatrix}$

Iteration 2, $C_{N2_i} := \min\left[\left(\frac{1\,\text{atm}}{\sigma'_{vo_i}}\right)^{0.784-0.0768\cdot\sqrt{\min\left(N_{1.60cs2_i}, 46\right)}}, 1.7\right]$ $\quad C_{N2} = \begin{pmatrix} 1.69 \\ 1.39 \\ 1.25 \\ 1.16 \\ 1.10 \\ 1.05 \end{pmatrix}$

Iteration 2, $N2_{1.60_i} := C_{N2_i} \cdot N_{60_i}$ $\quad N2_{1.60} = \begin{pmatrix} 84.35 \\ 73.93 \\ 70.87 \\ 73.57 \\ 58.34 \\ 66.25 \end{pmatrix}$ $\quad$ *Iteration satisfied since $N1_{1.60} = N2_{1.60}$

Iteration 3, $N_{1.60cs3} := N2_{1.60} + \Delta N = \begin{pmatrix} 89.96 \\ 79.54 \\ 76.48 \\ 79.18 \\ 63.95 \\ 71.86 \end{pmatrix}$

Iteration 3, $C_{N3_i} := \min\left[\left(\frac{1\,\text{atm}}{\sigma'_{vo_i}}\right)^{0.784-0.0768\cdot\sqrt{\min\left(N_{1.60cs3_i}, 46\right)}}, 1.7\right]$ $\quad C_{N3} = \begin{pmatrix} 1.69 \\ 1.39 \\ 1.25 \\ 1.16 \\ 1.10 \\ 1.05 \end{pmatrix}$

Iteration 3, $N3_{1.60_i} := C_{N3_i} \cdot N_{60_i}$ $\quad N3_{1.60} = \begin{pmatrix} 84.35 \\ 73.93 \\ 70.87 \\ 73.57 \\ 58.34 \\ 66.25 \end{pmatrix}$ $\quad$ *Iteration satisfied since $N2_{1.60} = N3_{1.60}$

Overburden correction factor, $C_N := C_{N3} = \begin{pmatrix} 1.69 \\ 1.39 \\ 1.25 \\ 1.16 \\ 1.10 \\ 1.05 \end{pmatrix}$

Corrected penetration resistance, $N_{1.60} := N3_{1.60} = \begin{pmatrix} 84.35 \\ 73.93 \\ 70.87 \\ 73.57 \\ 58.34 \\ 66.25 \end{pmatrix}$

Corrected penetration resistance in terms of equivalent clean-sand, $N_{1.60cs} := N_{1.60} + \Delta N = \begin{pmatrix} 89.96 \\ 79.54 \\ 76.48 \\ 79.18 \\ 63.95 \\ 71.86 \end{pmatrix}$

Overburden correction factor, $K_{\sigma_i} := \min\left(1 - \frac{1}{18.9 - 2.55\cdot\sqrt{\min\left(N_{1.60cs_i}, 37\right)}}\cdot\ln\left(\frac{\sigma'_{vo_i}}{1\text{atm}}\right), 1.1\right)$ $\quad K_\sigma = \begin{pmatrix} 1.10 \\ 1.10 \\ 1.10 \\ 1.10 \\ 1.10 \\ 1.05 \end{pmatrix}$

Cyclic resistance ratio adjusted to M = 7.5 and $\sigma`_v$ = 1 atm,

$$CRR_{M7.5\_1atm_i} := \min\left[e^{\left[\frac{N_{1.60cs_i}}{14.1} + \left(\frac{N_{1.60cs_i}}{126}\right)^2 - \left(\frac{N_{1.60cs_i}}{23.6}\right)^3 + \left(\frac{N_{1.60cs_i}}{25.4}\right)^4 - 2.8\right]}, 2\right] \qquad CRR_{M7.5\_1atm} = \begin{pmatrix} 2.00 \\ 2.00 \\ 2.00 \\ 2.00 \\ 2.00 \\ 2.00 \end{pmatrix}$$

Cyclic resistance ratio, $CRR_i := \min\left(CRR_{M7.5\_1atm_i} \cdot MSF \cdot K_{\sigma_i}, 2\right)$ $\quad CRR = \begin{pmatrix} 2.00 \\ 2.00 \\ 2.00 \\ 2.00 \\ 2.00 \\ 1.99 \end{pmatrix}$

Shear stress reduction parameter,

$$\alpha(z) := -1.012 - 1.126 \cdot \sin\left(\frac{\frac{z}{m}}{11.73} + 5.133\right)$$

$$\beta(z) := 0.106 + 0.118 \cdot \sin\left(\frac{\frac{z}{m}}{11.28} + 5.142\right)$$

$$r_d := e^{(\alpha(z)+\beta(z)\cdot M)} = \begin{pmatrix} 1.00 \\ 0.99 \\ 0.98 \\ 0.96 \\ 0.95 \\ 0.93 \end{pmatrix}$$

Total overburden stress, $\sigma_{voe_i} := \begin{vmatrix} \gamma_a \cdot z_i \ \text{ if } z_i \le GWT_e \\ \gamma_a \cdot GWT_d + \gamma_b \cdot \left(z_i - GWT_e\right) \ \text{ otherwise} \end{vmatrix}$ $\quad \sigma_{voe} = \begin{pmatrix} 13.89 \\ 42.73 \\ 71.70 \\ 100.67 \\ 129.64 \\ 158.60 \end{pmatrix} \cdot kPa$

(at time of earthquake)

Effective overburden stress, $\sigma'_{voe_i} := \begin{vmatrix} \gamma_a \cdot z_i \ \text{ if } z_i \le GWT_e \\ \gamma_a \cdot GWT_e + \left(\gamma_b - \gamma_w\right) \cdot \left(z_i - GWT_e\right) \ \text{ otherwise} \end{vmatrix}$ $\quad \sigma'_{voe} = \begin{pmatrix} 13.89 \\ 29.28 \\ 43.31 \\ 57.33 \\ 71.36 \\ 85.38 \end{pmatrix} \cdot kPa$

(at time of earthquake)

Earthquake-induced cyclic stress ratio, $CSR_i := 0.65 \cdot \frac{\sigma_{vo_i}}{\sigma'_{vo_i}} \cdot \frac{a_{max}}{g} \cdot r_{d_i}$ $\quad CSR = \begin{pmatrix} 0.17 \\ 0.25 \\ 0.28 \\ 0.29 \\ 0.30 \\ 0.30 \end{pmatrix}$

Factor of safety against liquefaction triggering, $FS := \frac{CRR}{CSR} = \begin{pmatrix} 11.55 \\ 8.00 \\ 7.14 \\ 6.84 \\ 6.72 \\ 6.67 \end{pmatrix}$

Summary of results

Depth	Measured Blow Count	Fines Content	Cyclic Resistance Ratio	Cyclic Stress Ratio	Factor of safety against liquefaction triggering
$D = \begin{pmatrix} 5.00 \\ 10.00 \\ 15.00 \\ 20.00 \\ 25.00 \\ 30.00 \end{pmatrix} ft$	$N_m := \begin{pmatrix} 50 \\ 50 \\ 50 \\ 50 \\ 42 \\ 50 \end{pmatrix}$	$FC := \begin{pmatrix} 46 \\ 46 \\ 46 \\ 58 \\ 58 \\ 58 \end{pmatrix} \%$	$CRR = \begin{pmatrix} 2.00 \\ 2.00 \\ 2.00 \\ 2.00 \\ 2.00 \\ 1.99 \end{pmatrix}$	$CSR = \begin{pmatrix} 0.17 \\ 0.25 \\ 0.28 \\ 0.29 \\ 0.30 \\ 0.30 \end{pmatrix}$	$FS = \begin{pmatrix} 11.55 \\ 8.00 \\ 7.14 \\ 6.84 \\ 6.72 \\ 6.67 \end{pmatrix}$

APPENDIX F.4C

Deep Seated Slope Stability (Global) Analysis (Rev 3)

PURPOSE

The purpose of this analysis is to evaluate deep-seated (global) slope stability of the proposed Class II Disposal Facility at Scepter, Inc. in Waverly, Tennessee. The analysis is provided pursuant to Tennessee Department of Environment and Conservation (TDEC) rule 400-11-01-.04(2)(a)3 which states the facility must be located, designed, constructed, and maintained, and closed in such a manner as to minimize to the extent practicable the potential for releases of solid waste, solid waste constituents, or other potentially harmful material to the environment except in a manner authorized by state law. Slope stability analysis is performed to identify potential slope failure mechanisms which could release waste into the environment and determine the factor of safety against slope failure through the waste and/or foundation and along the liner.

I. SITE AND PROJECT DESCRIPTION

The static deep-seated stability analysis was performed as part of the industrial waste permit application for the Scepter Site located in Waverly, Tennessee.

The proposed site will be permitted as a new industrial solid waste landfill through TDEC Division of Solid Waste Management.

The following sections summarize the methodology, assumptions, and results of the global slope stability analysis in the facility area. For further detail on the specific calculations performed, refer to the corresponding data provided in the **Attachments**.

II. MODEL THEORY

All global stability analyses were performed using Geoslope, Inc.'s geotechnical analysis software, SLOPE/W, specifically designed for evaluating slope stability. This software is a tool used to evaluate slope stability based on various limit equilibrium methods. Limit equilibrium methods are the most popular techniques used for slope stability studies and generally consist of cutting the slope into fine vertical slices and then evaluating each slice for static equilibrium in terms of forces and/or moments as depicted in **Figure 1** below. For this evaluation, Spencer's Method was selected in SLOPE/W to evaluate slope stability and determine slip surface factors of safety (FS) for circular and block (translational) failure geometries. A solution by Spencer's Method involves an iterative, trial and error procedure in which values for the factor of safety and side force inclination are assumed until force and moment equilibrium are satisfied for each slice. This method provides an accurate procedure applicable to most slope geometries and soil profiles and represents the simplest complete equilibrium procedure for computing factors of safety. After inputting information for the cross section geometry, soil material characteristics, and groundwater conditions, the software computes safety factors for a large number of trial surfaces and the slip surface with the lowest safety factor can be viewed graphically.

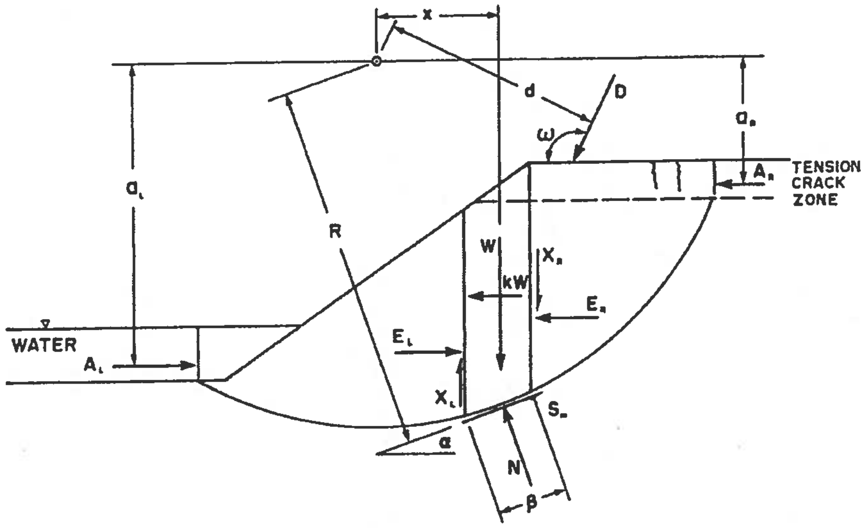


Figure 1. Limit Equilibrium Model

Where:

W = Total weight of a slice of width b and height h

N = Total normal force on the base of the slice

S = Shear force mobilized on the base of each slice

E = Horizontal interslice normal forces

X = Vertical interslice shear forces

D = An external line load

k_W = Horizontal seismic load applied through the centroid of each slice

R = Radius of circular slip surface or moment arm associated with the mobilized shear force, S_m for any shape of slip surface

f = Perpendicular offset of the normal force from the center of rotation or from the center of slope

x = Horizontal distance from the centerline of each slice to the center of rotation or to the center of moments

e = Vertical distance from the centroid of each slice to the center of rotation or to the center of moments

d = Perpendicular distance from a line load to the center of rotation or to the center of moments

h = Vertical distance from the center of the base of each slice to the uppermost line in the geometry

a = Perpendicular distance from the resultant external water force to the center of rotation or to the center of moments

A = Resultant external water forces

w = Angle of the line load from the horizontal. This angle is measured counter-clockwise from the positive x-axis

a = Angle between the tangent to the center of the base of each slice and the horizontal

For this analysis, the following conditions and target minimum factors of safety were used based on the anticipated probability and consequences of failure:

- Static conditions: FS ≥ 1.50;
- Static conditions (residual strength): FS ≥ 1.20; and
- Seismic conditions: FS ≥ 1.00.

The static/peak strength condition corresponds to a long term condition. Typically slope stability analyses require a static factor of safety between 1.4 and 2.0 (or potentially higher if potential unknown conditions warrant it). A factor of safety of 1.5 is commonly used for landfill cap and liner system design for long term static conditions. Although mobilizing residual strength in the engineered components is unlikely to occur, the design was analyzed for residual strength conditions where applicable. Because this condition is unlikely, a lower factor of safety is warranted.

The following information was used to develop the stability models in SLOPE/W:

- Model geometry (slope characteristics and cross section location);
- Subsurface conditions (soil stratigraphy and groundwater and/or leachate levels); and
- Material properties (soil and landfill construction materials).

III. MODEL GEOMETRY

The model geometry includes information concerning the slope characteristics and cross section locations for evaluation.

a. Slope Characteristics

Proposed excavated slopes in natural materials have a maximum slope of 3H:1V. Maximum depth of excavation is approximately 455 feet and occurs in Phase 1.

Final slopes are comprised of a composite grade that are maximum of 3H:1V between surface water drainage swales. The overall final grade from toe to top is at a maximum of 3H:1V with a maximum bench height of 30 feet and a maximum elevation of 645.75 ft.

b. Cross Section Location

Four cross sections were selected to represent the final slopes at the proposed Waverly site. Cross sections were chosen based on the following criteria:

- Height and steepness of exterior and bottom liner slopes;
- Depth of excavation and direction of bottom liner slope; and
- Strength of subsurface soil material.

Based on the criteria noted above, the following cross sections were chosen for analysis under final conditions:

- Cross Section A-A'
- Cross Section B-B'
- Cross Section C-C'
- Cross Section D-D'

A plan view showing the cross section locations is provided in **Attachment A.** Each individual cross section can be viewed in the stability outputs provided in **Attachments B, C, D, and E** at the end of this report.

IV. SUMMARY OF SUBSURFACE CONDITIONS

To perform the slope stability analysis, a hydrogeologic exploration program was performed recently consisting of 7 borings and 2 monitoring wells within and around the proposed footprint of the facility and constituting approximately 494 feet of drilling. In addition, previously developed subsurface data within and in the vicinity of the proposed landfill footprint including over 15 additional borings and laboratory test data were utilized in the assessment. Detailed observations, soil sample test results, analyses, and conclusions concerning the subsurface conditions are presented in the Hydrogeologic Exploration report provided with this permit. Additional information was obtained from a subsurface exploration performed by GA Technical Services in 1989. Finally, previous stability analyses performed for different phases of the Scepter Landfill in 2008 and 2012 were reviewed and considered in assessing slope stability. The following summarizes the in-situ conditions related to the stability analysis.

a. Soil and Bedrock

The subsurface soil material generally consists of residuum composed of low to medium plasticity clay (CL), clayey silt (ML), clayey sand (SC), and silty sand (SM) based on the Unified Soil Classification System (USCS). Laboratory tested samples obtained from the site typically had between 27 to 73 percent of material by weight passing the #200 sieve with 20% or more finer than 0.005 mm. The plasticity index of the soils generally ranged between 5 and 15 percent indicating a cohesive material of low to medium plasticity. Gravel content varied from 2.7% to 13.7% in the fine grained materials to 2.3% to 39.7% in the coarse grained materials. When corrected for gravel content, the effective percent fines of the samples tended to increase by 10% to 40%. As a result, it is expected that the majority of the overburden will behave as a cohesive material regardless of USCS classification. Natural moisture typically varied between 6 and 41 percent. Hydraulic conductivity test results varied from 4.7×10^{-7} cm/s to 7.5×10^{-7} cm/s for samples obtained from relatively undisturbed Shelby tubes.

Generally, weathered bedrock immediately underlies the unconsolidated soil material at the site. Bedrock was not encountered during the most recent hydrogeologic investigation. However, depth to bedrock was determined in 1988 during the installation of a well water supply at the Scepter facility. According to well installation logs, the black shale of the Chattanooga Shale was encountered at approximately 110 to 130 feet below ground surface (bgs) (roughly 270 feet msl). The uppermost bedrock is comprised primarily of the Fort Payne formation overlying the Chattanooga shale. These upper units are generally described as follows:

- The Fort Payne formation in the area constitutes the upper 100 to 200 feet of the ridges. This formation has weathered almost completely to a residuum of a chert silt/clay matrix and colluvium.
- The Chattanooga Shale is black, fissile, pyritic, carbonaceous often called bituminous shale which acts as a confining unit. The Chattanooga shale does not outcrop at the site and was encountered during drilling for the Scepter facility supply well at a depth of approximately 130 feet below ground surface (bgs).

b. Groundwater

Groundwater generally occurs in two zones at the site: shallow systems in the alluvial deposits beneath the area at depths ranging from approximately 35 to 80 feet bgs and the deep regional system within the carbonate aquifer in the Fort Payne formation.

The shallow water is semi-confined and its connection to the surface (i.e., recharge by surface infiltration) is evidenced by temperature and water level fluctuations in wells shortly following precipitation events. Local groundwater flow typically mimics the topography, and the flow direction is generally toward area drainage features. A topographically low area is present north of the Scepter landfill.

No ponds, springs, sinkholes, ephemeral seeps or wetlands were identified during site reconnaissance. A perennial water feature exists on the north side of the CSX railroad tracks which is identified as a blue line stream within Leach Hollow on a USGS topographic map. This is the receiving stream for all site drainage which flows west to the Tennessee River.

The regional system in the vicinity of the sites is located within the carbonate aquifer. Groundwater within the regional aquifer is encountered in the Fort Payne formation. Recharge to the groundwater occurs primarily from infiltration of precipitation. The recharge waters ultimately migrates downgradient toward the Tennessee River. The Tennessee River flows south to north adjacent to the Scepter site. It is dammed to form Kentucky Lake near Lake City, Kentucky.

a. Stratigraphy

Based on the results of the subsurface exploration program and the design of the facility, the stratigraphic model was developed for the stability analysis. Beginning from the top, the stratigraphy was modeled with the following layers.

Table 1. Global Stability Model Stratigraphy

Layer	Soil Layer	Thickness	Description
1	Cap System	3.5 ft	This layer includes the proposed topsoil, cap cover soil, geocomposite drainage layer, flexible membrane liner layer, geocomposite gas pressure relief layer, barrier soil, and intermediate cover soil. These layers are modeled as a single composite layer with conservative unit weight and shear strength properties.
2	Waste Material	Varies	This layer includes the aluminum salt cake waste material produced as a byproduct of the aluminum refining process. Strength parameters are derived from its similarity in particle size and texture to sand.
3	Liner System	2 ft	This layer includes the protective cover soil, protective cover sand, geocomposite drainage layer, and flexible membrane liner. These layers are modeled as a single composite layer with conservative unit weight and shear strength properties.
4	Clay Soil	2 ft	This layer is composed of recompacted soil with permeability equal or less than 10^{-7} cm/sec to construct a compacted clay liner above the geologic buffer/native soils. It is expected for this layer to be constructed from on-site materials.
5	Foundation Soils	Varies	This layer is composed of native residuum/alluvial material identified as clay to silty sand. This soil describes the entirety of the on-site soils. Hydraulic conductivity testing of relatively undisturbed samples resulted in acceptable permeability values for a geologic buffer. Therefore, the upper 10 feet of the native soil below cut elevations will be considered a geologic buffer.
6	Bedrock	Not modeled	Bedrock was not modeled due to its relatively great depth below the site.

b. Water Table

Groundwater levels at the temporary piezometer and the well locations were manually measured periodically using a water level meter. The water levels were converted to elevation data using the surveyed top-of-casing reference elevation. This data is presented in the Hydrogeological Report for the site. In general, the potentiometric water surface exists approximately two feet below the proposed cut elevations.

V. MATERIAL PROPERTIES

The following sections describe the values established for each of the layers used in the model for global stability.

Cap System

The cap system layer is modeled as a composite of materials. This layer includes 6 inches of vegetative cover, 18 inches of protective cover layer, a geocomposite drainage layer, a flexible membrane liner, a geocomposite gas pressure relief layer, 6 inches of barrier soils, and a foot of intermediate cover soils. The geosynthetic layers are thin such that their weight is negligible. Only the soil layers contribute substantially to the unit weight of these materials. A density of 110 pcf is conservatively assumed for this analysis.

Typically, one or more of the components comprising this layer will be a critical failure surface in the stability analysis. This surface could pass entirely through one or more of any of the components or along an interface between geosynthetic components such as the geocomposite drainage layer and the FML. Consequently, the strength of each of the components and their respective interfaces must all, at a minimum, exhibit a minimum shear strength to meet the required factors of safety in the stability analysis.

Therefore, given that this layer is well drained, peak effective stress parameters for the cap system layer were assigned based on published data contained in GRI Report #30 (Direct Shear Database of Geosynthetic to Geosynthetic and Geosynthetic to Soil Interfaces-Geosynthetic Research Institute, June 14, 2005). A copy of the GRI Report #30 published data is provided in **Attachment H.**

These published values were used to the global stability of the landfill with particular respect to a translational failure along the liner. The published values selected for use in the analysis were the lowest overall shear strength interface of the liner system (LLDPE textured flexible membrane liner on geocomposite). These input parameters obtained from GRI #30 for the global stability analysis final cover layer provided in **Table 2** below.

Table 2. Global Stability Analysis Input Parameters – Intermediate/Final Cover Layer

Layer	Total Unit Wt. γ_T (pcf)	Peak Effective Stress Parameters		Residual Strength Parameters	
		ϕ'_P (deg.)	C'_P (psf)	ϕ'_R (deg.)	C'_R (psf)
Final Cover	110	26	169	17	198

Waste Materials

The waste disposed at the Scepter Waverly landfill is an aluminum salt cake material produced as a byproduct of the aluminum refining process. AECOM does not have laboratory test data but understands the waste material is granular in nature at point of placement, with particles sizes being variably graded and varying from boulder and cobble sized fragments to fine sands. Based on the variability of particle size, void space between larger particles would be expected, leading to a lower overall unit weight. On-site, native soil (conservatively assessed below as having an angle of internal friction of 32.5 degrees in the drained condition) is used as daily cover for the waste material. Therefore, the unit of the waste is assigned to be 110 pcf, considering the mixture of daily cover (120 pcf) and waste material. Further, an angle of internal friction of 32 degrees was conservatively assigned based on the granular nature of the waste material and of the daily cover.

Liner System

The liner and leachate collection system layer is modeled as a composite of materials. This layer includes the protective cover layers of sand and clay, geocomposite drainage layer, flexible membrane liner (FML), compacted clay soil, and geologic buffer. Only the protective layers and the geologic buffer contribute substantially to the unit weight of these materials. Total unit weight for the sand and clay protective layers is expected to vary from 110 to 120 pcf depending on the confining stress and moisture content. It is anticipated that the material will be closer to its maximum density as the facility is developed; however, for this analysis a lower density is conservatively assumed.

Typically, one or more of the components comprising this layer will be a critical failure surface in the deep seated stability analysis. This surface could pass entirely through one or more of any of the components such as the FML or along an interface between geosynthetic components such as the geocomposite drainage layer and the FML. Consequently, the strength of each of the components and their respective interfaces must all, at a minimum, exhibit a minimum shear strength to meet the required factors of safety in the stability analysis.

Accordingly, peak effective shear strength parameters and residual shear strength parameters for the liner/drainage layer were assigned based on published data contained in GRI Report #30 (Direct Shear Database of Geosynthetic to Geosynthetic and Geosynthetic to Soil Interfaces-Geosynthetic Research Institute, June 14, 2005). These published values were used to evaluate the global stability of the landfill with particular respect to a translational failure along the liner. The published values selected for use in the analysis were the lowest overall shear strength interface of the liner system (in this case HDPE textured flexible membrane liner on geocomposite). These input parameters obtained from GRI #30 for the global stability analysis composite liner/drainage/protective layer are provided in **Table 3** below. A copy of the GRI Report #30 published data is provided in **Attachment H.**

Table 3. Global Stability Analysis Input Parameters – Liner/Drainage/Protective Layer Materials-GRI#30 Data

Layer	Total Unit Wt. γ_T (pcf)	Peak Effective Stress Parameters		Residual Strength Parameters	
		ϕ'_P (deg.)	c'_P (psf)	ϕ'_R (deg.)	c'_P (psf)
Liner/Drainage/Protective-HDPE-T/Geocomposite	110	26	0	15	0

Because the geosynthetic materials are not typically available for testing to confirm strength estimates at the time of stability analysis, the values in **Table 4** were back calculated based on the minimum acceptable factors of safety from the slope stability analysis. Subsequently, these values were evaluated to confirm they represent a reasonable conservative estimate of the minimum expected shear strength for all the components comprising this layer. The minimum shear strengths of each of the components and their respective interfaces should all be tested utilizing representative samples of those materials to confirm that the minimum shear strength estimate is valid. The procedures for this testing are outlined in Section 4 and Table 2 of the Construction Quality Assurance Plan (Part D of the Permit Application). Each interface will be tested at a range of normal loads to identify the critical interface and confirm that the materials used for construction will result in adequate factors of safety in accordance with the global stability analysis presented herein.

Table 4. Global Stability Analysis Input Parameters – Liner/Drainage/Protective Layer Materials-Minimum Required Data

Layer	Total Unit Wt. γ_T (pcf)	Peak Effective Stress Parameters		Residual Strength Parameters	
		ϕ'_P (deg.)	c'_P (psf)	ϕ'_R (deg.)	c'_P (psf)
Liner/Drainage/Protective	110	21.9	0	8.1	0

Compacted Soil Liner/Structural Fill

To assess shear strength parameters of the compacted soil liner, Table 1-"Typical Properties of Compacted Soils", page 7.2-39, from the Naval Facilities Engineering Command (NAVFAC), Foundations and Earth Structures, DM 7.2, was utilized. Specifically, the compacted soil liner as placed is consistent with the soil type "Inorganic clay of low to medium plasticity" from Table 1, and a cohesion of 270 psf and angle of internal friction of 28 degrees were utilized as a result. A copy of Table 1 is provided in **Attachment I**.

Foundation Soils

The material properties of the foundation soil were determined based on a previous subsurface exploration and laboratory testing data. During the previous subsurface exploration, site soils encountered consist of low to medium plasticity clay (CL), clayey silt (ML), clayey sand (SC), and silty sand (SM) based on the Unified Soil Classification System (USCS). Although a heterogeneous combination of fine grained and coarse grained materials were encountered and classified based on the laboratory testing, the foundation soils were modeled as one material based on the irregularity of lithological changes through the overburden, overall quantity of gravel, and overall quantity of fine sized particles.

The unit weights used in the stability modeling for the foundation material were estimated based on permeability laboratory testing results from the Hydrogeologic Study performed in 2016 for the proposed East Phase expansion of the existing facility. To assess shear strength multiple correlations relating the soil's Atterberg Limits, N-values (blow counts), and particle size distribution were performed on historical samples. Specifically, the following correlations were used to estimate shear strength of the foundation materials:

(Kenney 1959)	$\phi' = 0.0031 * PI^2 - 0.4433\ PI + 38.939$
(USACE 1976)	$\phi' = -0.0018 * PI^2 - 0.1465\ PI + 32.029$
(USACE WWES 1962)	$\phi' = 0.0042 * PI^2 - 0.5565\ PI + 37.306$
(Peck 1974)	$\phi' = -0.0016\ N^2 + 0.3661\ N + 26.468$
(Ladd 1977)	$\phi' = \mathrm{ARCSIN}(1-K_o)$

For the effective condition, cohesion was conservatively taken to be 0 psf. An effective angle of internal friction ϕ' was calculated for each sample subjected to laboratory testing and all correlated strengths were charted. A total of 35 samples were assessed from 12 different exploratory borings performed on the Scepter property. The individual ϕ' values varied from 27 to 46.9 degrees. To select a design strength, the data was sorted, and a ϕ' value was chosen such that 2/3 of the values were higher and 1/3 were lower. Specifically, the data sort resulted in a ϕ' of 32.5 degrees. The results of the correlations are shown below in **Figure 2**.

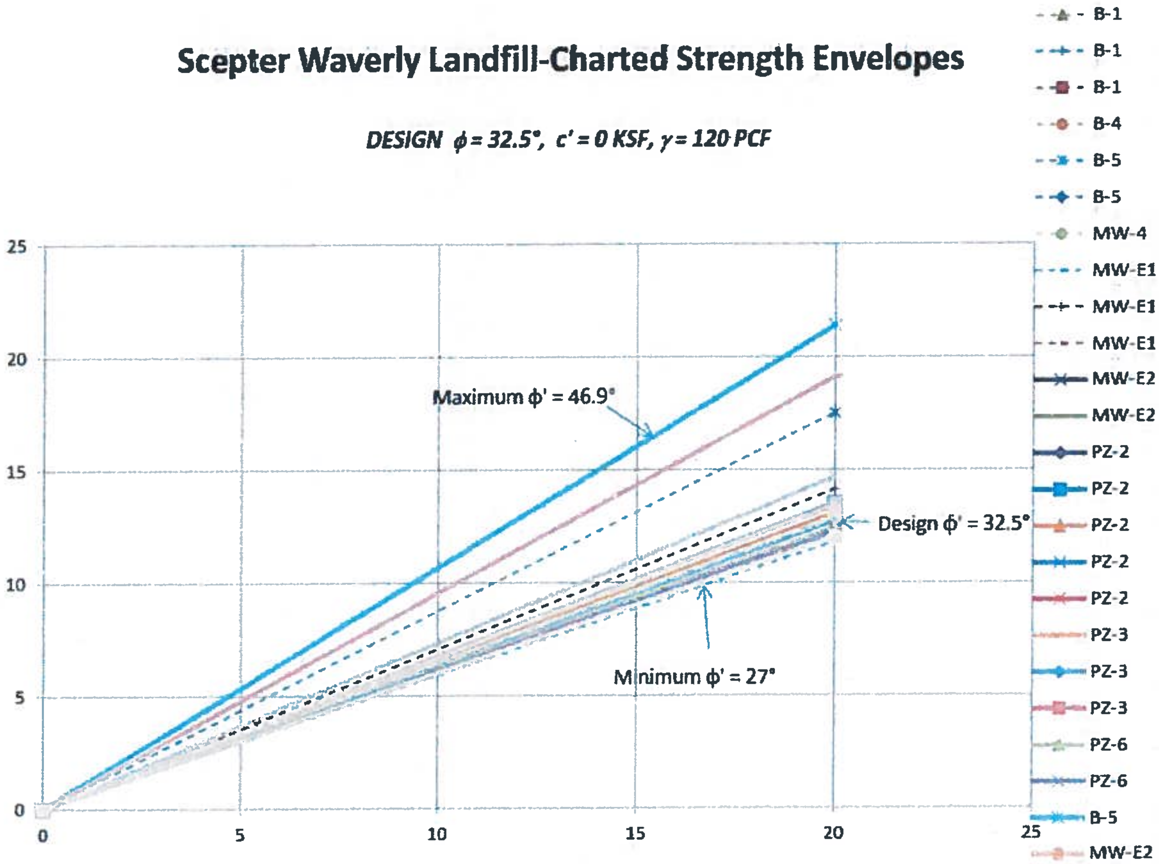


Figure 2: Charted Strength Envelopes

Hydraulic conductivity testing of foundation (residual) soils yielded values considered acceptable for use as geologic buffer material. Therefore, the upper 10 feet of in-situ foundation soil is considered to be the geologic buffer for the landfill.

VI. SEISMIC LOADS

The seismic coefficient (k_S) is calculated based on the seismic hazard identified at the site. It is a variable in which the inertia forces due to earthquake shaking are represented by a constant horizontal force equal to the weight of the potential sliding mass multiplied by the peak average acceleration of the failure mass. This additional force is used in the limit equilibrium stability analyses to account for seismic impacts in the design for the facility and minimize impacts to the engineered components. This approach is commonly called a pseudostatic analysis and is one of the simplest means used in earthquake engineering to analyze the seismic response of soil embankments and slopes. The seismic coefficient is activated in the slope stability model to perform the pseudostatic analysis. Field observations indicate that the pseudostatic method is appropriate for evaluating the performance of embankments constructed of fill materials that do not lose significant strength during earthquakes such as clays, clayey soils, dry or moist cohesionless soils, and dense cohesionless soils (Seed, 1985).

Based on the seismic coefficient calculations provided separately, the seismic coefficient for use in the pseudostatic analysis is shown in **Table 5** below.

Table 5. Global Stability Analysis Input Parameters – Seismic Coefficient

Location	Seismic Coefficient*
K_S	0.266g
$K_{S\text{-}YIELD}$	0.35g

*Peak ground acceleration in the bedrock is 0.266g as determined from the USGS seismic hazard maps for the region for 2% PE in 50 years. $K_{S\text{-}LINER}$ is the representative seismic response for the liner system at the critical cross section based on the analysis in the seismic coefficient calculation where deformation is negligible (less than 20 mm or 10 mm, respectively.)

To evaluate the seismic stability, the maximum seismic coefficient necessary to cause instability is determined by calculating the seismic coefficient where FS=1.0. This is the yield coefficient (K_{YIELD}). If K_{YIELD} is equal to or greater than K_S, then the slope is stable under seismic load.

VII. ANALYSIS SCENARIOS

The following scenarios were evaluated for slope stability of the landfill. These scenarios were selected to model possible failure mechanisms.

Scenario A: Considers a large rotational slip surface through the waste material during static conditions.

Scenario B: Considers a block (translational) slip surface along the liner during static conditions.

Scenario C: Considers a large circular slip surface through the waste material during seismic conditions. A horizontal seismic load was applied to the slope based on the peak ground acceleration of 0.266 g obtained from USGS hazard maps for 2% probability of exceedance in 50 years.

Scenario D: Considers a block (translational) slip surface along the liner during seismic conditions. A horizontal seismic load was applied to the slope based on the peak ground acceleration of 0.266 g obtained from USGS hazard maps for 2% probability of exceedance in 50 years.

Scenario E: Considers a block (translational) slip surface along the liner during static conditions using residual strength parameters for the liner system.

Scenario F: Considers a shallow rotational slip surface through the waste material during static conditions. The analysis assumes the cap system has not been installed yet.

Scenario G: Considers a large rotational slip surface through the foundation material during static conditions.

VIII. SUMMARY OF RESULTS

A summary of the critical slope stability analysis results is provided in **Attachment F**. Detailed slope stability analysis output from Slope/W is provided in **Attachments B, C, D, and E**. The resulting critical factors of safety (FS) for each scenario described in Section IV are summarized in **Table 6**.

Table 6. Slope Stability Analysis Results

Load Condition/ Failure Type	Cross Section A-A'	Cross Section B-B'	Cross Section C-C'	Cross Section D-D'	Minimum Required Factor of Safety
A/Static Rotational through Waste	2.40	2.36	2.39	2.65	1.50
B/Static Translational along Liner	2.47	3.43	5.16	3.36	1.50
C/Seismic Rotational through Waste	1.20	1.20	1.22	1.26	1.00
D/Seismic Translational along Liner	1.12	1.33	1.54	1.28	1.00
E/Static Translational along Liner (Residual)	1.68	2.90	4.83	2.29	1.20
F/Shallow Rotational through Waste	2.40	2.31	2.33	2.64	1.50
G/Static Rotational through Foundation	2.42	3.09	3.40	2.91	1.50

The stability outputs for cross sections A-A', B-B', C-C', and D-D' are provided in **Attachments B, C, D, and E,** respectively. The results show that cross-section A-A' (see **Attachment F** for the critical cross section details) is the critical cross section.

- The minimum peak shear strength required for this layer is represented by a Mohr failure envelope equivalent to a cohesion/adhesion value equal to zero and a minimum friction angle (ϕ') of 21.9 degrees controlled by final conditions represented by the West cross section.

- Similarly, the minimum residual shear strength is equivalent to a cohesion/adhesion value equal to zero and a minimum friction angle of 8.1 degrees and is controlled by the West cross section.

Other acceptable effective strength parameters can also meet the minimum shear strength requirements to have an acceptable factor of safety. A chart depicting the set of ϕ' and c' combinations that satisfy the requirements for the minimum factor of safety for the proposed geosynthetic interfaces analyzed and the resulting curve is shown in **Attachment J**. This chart can be used to determine acceptable shear strength values resulting from interface friction testing performed to prequalify the geosynthetic material used for constructing the bottom liner and leachate collection system.

IX. CONCLUSIONS

Given the assumptions and scenarios discussed in the preceding sections, the stability analysis shows that the minimum factors of safety for the site are maintained based on the site design. Minimum shear strength parameters assumed for the engineered components including the geologic buffer/compacted soil liner and geosynthetic components should be tested and prequalified in accordance with the project CQA plan before being installed.

X. REFERENCES

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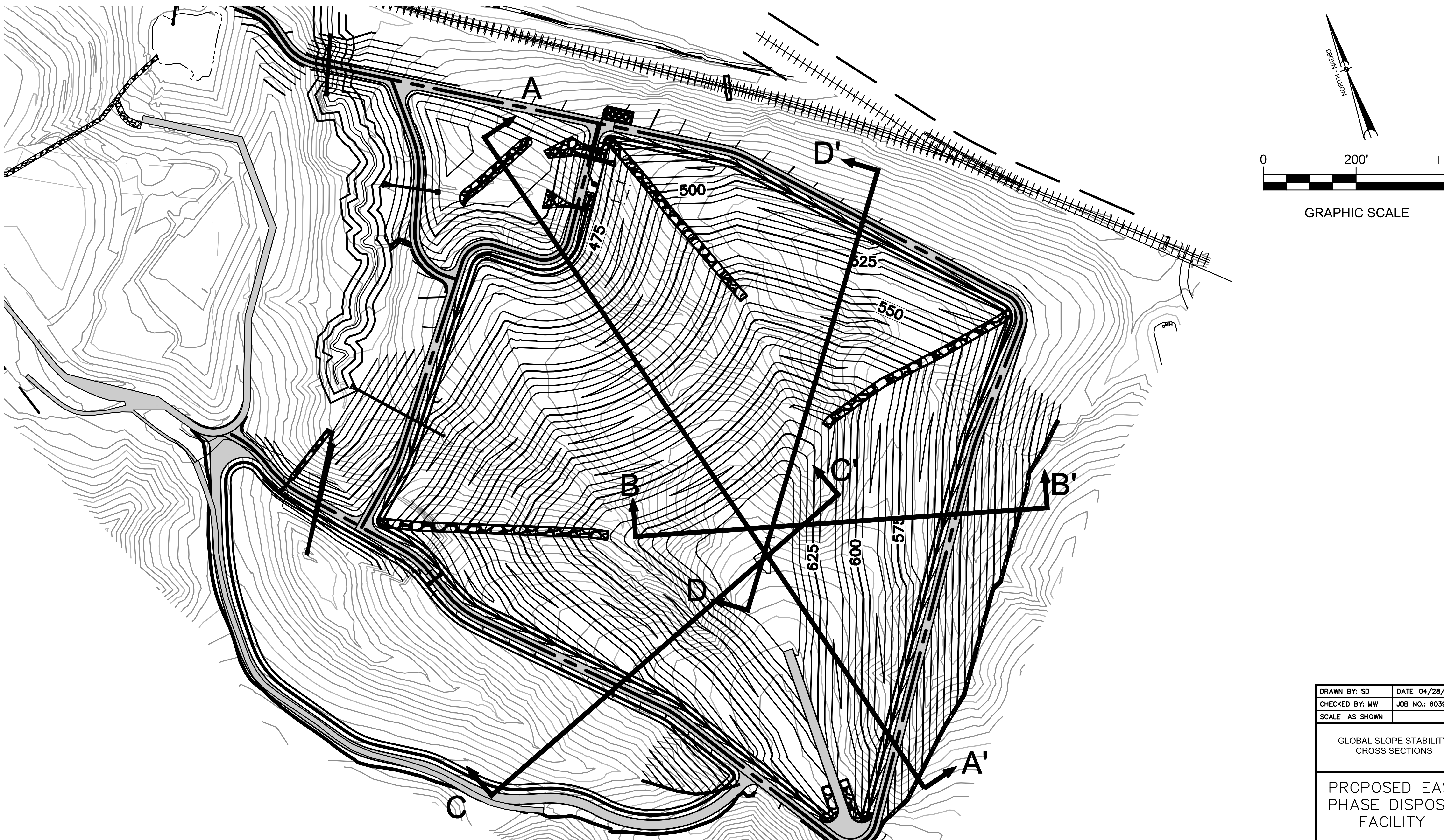
XI. ATTACHMENTS

Attachment A	Plan View of Cross Sections
Attachment B	Cross Section A-A' Slope Stability Output
Attachment C	Cross Section B-B' Slope Stability Output
Attachment D	Cross Section C-C' Slope Stability Output
Attachment E	Cross Section D-D' Slope Stability Output
Attachment F	Critical Cross Section
Attachment G	Charted Shear Strength Envelope for Foundation Soils
Attachment H	GRI Report #30 Published Values
Attachment I	NAVFAC Table 1, page 7.2-39, DM 7.2
Attachment J	Zone of Acceptable Values

Attachment A

Plan View



NOTE:
FINAL PROPOSED CAP GRADES AND EXISTING SITE GRADES ARE SHOWN IN PLAN VIEW.

DRAWN BY: SD	DATE 04/28/2017
CHECKED BY: MW	JOB NO.: 60398526
SCALE AS SHOWN	

GLOBAL SLOPE STABILITY
CROSS SECTIONS

PROPOSED EAST PHASE DISPOSAL FACILITY

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

ATTACHMENT A

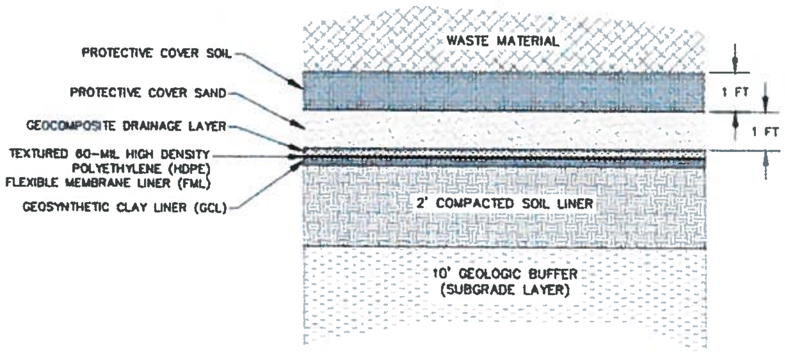
S:\2016\Scepter\028 Waverly\02 Landfill\Design\Calculations\Geotechnical\2016\CRITICAL CROSS SECTION.dwg User:DouangvilayS Sep 19, 2017 – 11:45am

Attachment B Cross Section A-A'

Slope Stability Output

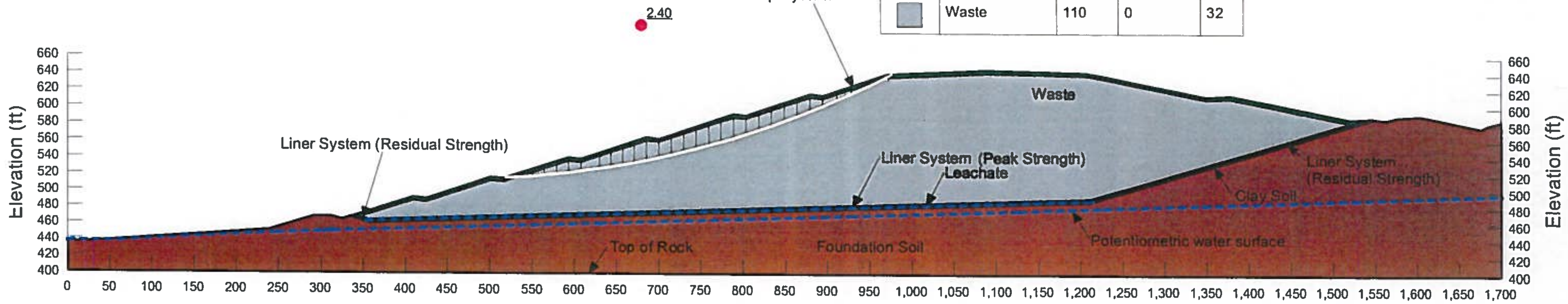
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Critical Section A-A'

Scenario A - Static
Slip surface: Large circular through waste
Factor of Safety: 2.40



Liner detail from AECOM permit drawing P-17.

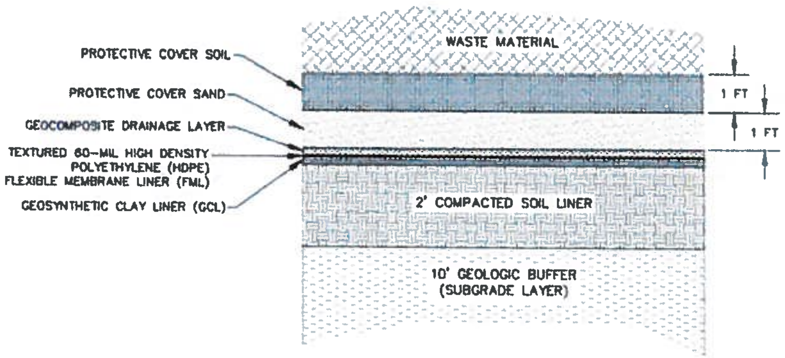
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

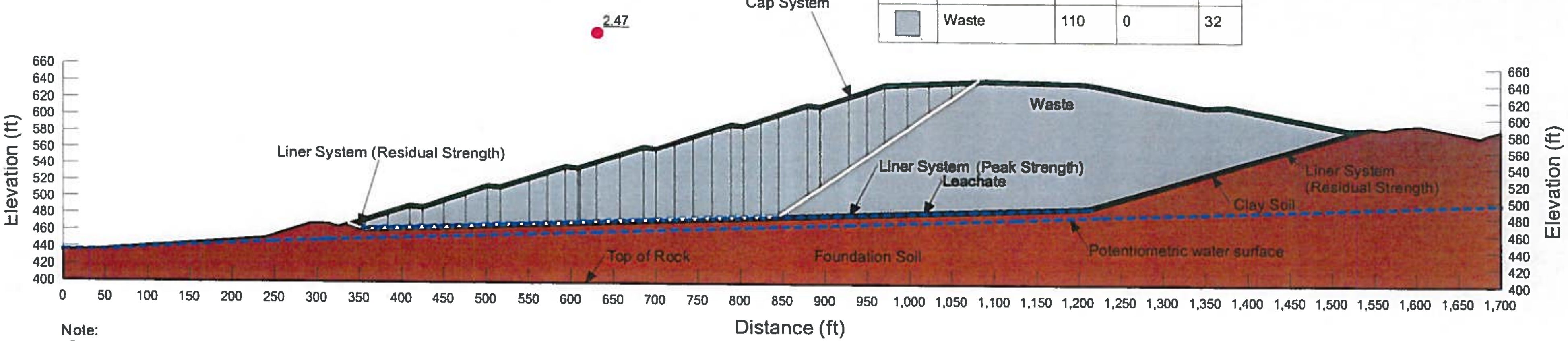
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Critical Section A-A'

Scenario B - Static
Slip surface: Translational along liner
Factor of Safety: 2.47



Liner detail from AECOM permit drawing P-17.

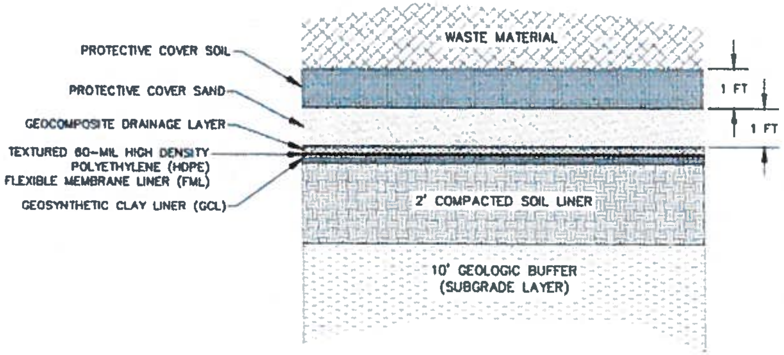
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

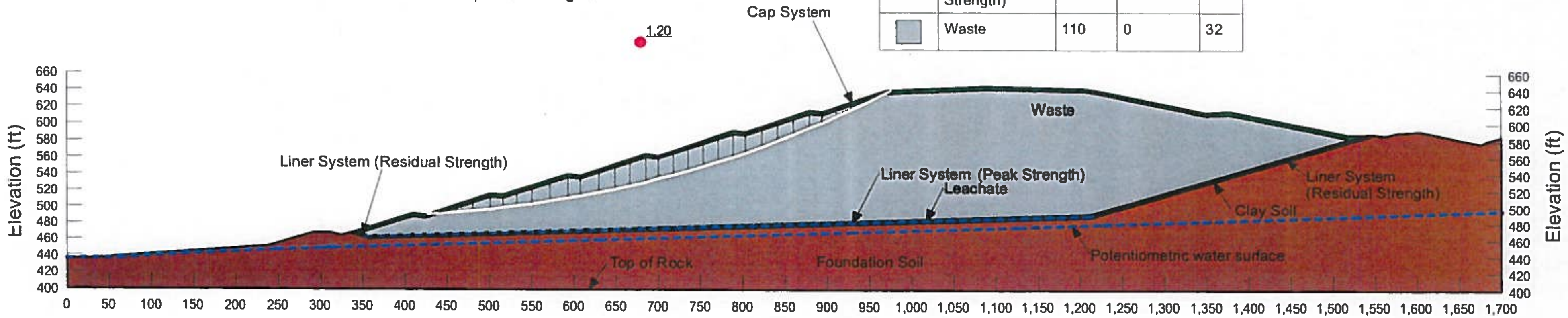
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Critical Section A-A'

Scenario C - Pseudostatic
Slip surface: Large circular through waste
Factor of Safety: 1.20
Horizontal Seismic Coefficient: 0.266g



Liner detail from AECOM permit drawing P-17.

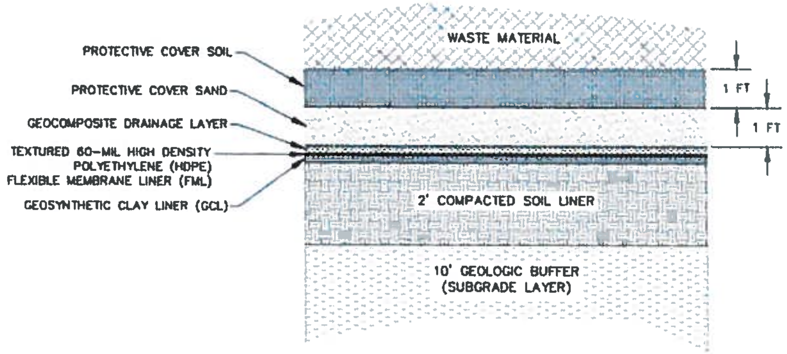
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

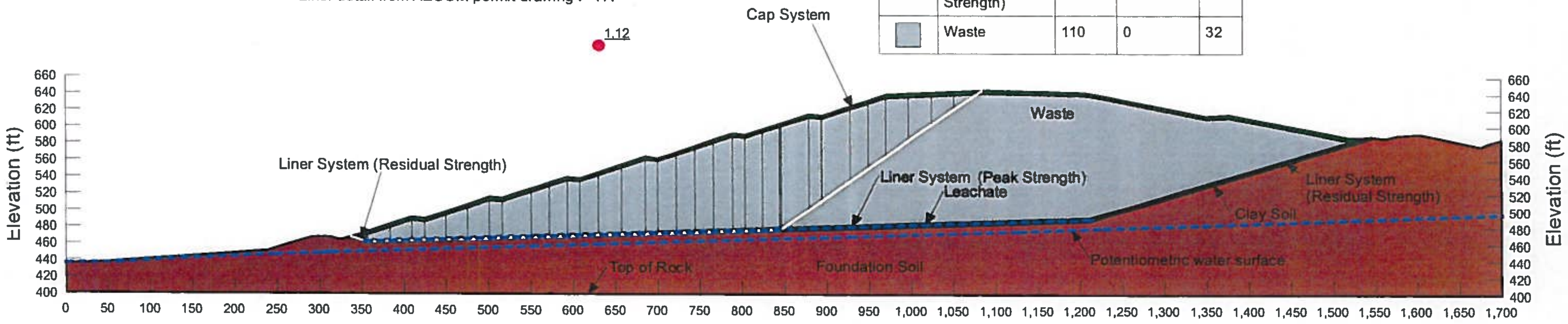
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Critical Section A-A'

Scenario D - Pseudostatic
Slip surface: Translational along liner
Factor of Safety: 1.12
Horizontal Seismic Coefficient: 0.266g



Liner detail from AECOM permit drawing P-17.

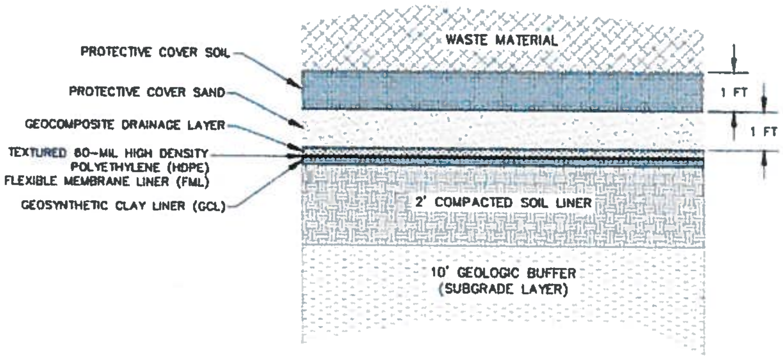
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

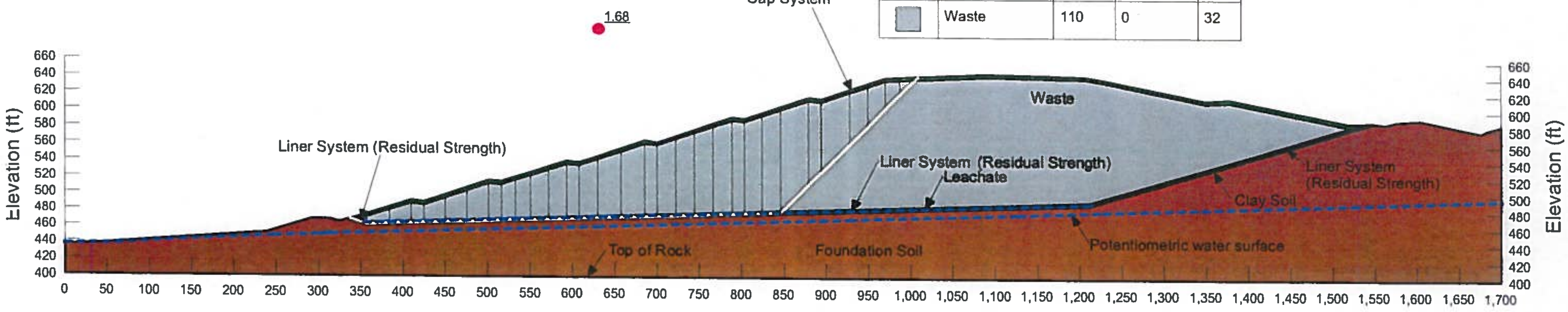
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Critical Section A-A'

Scenario E - Static
Slip surface: Translational along residual liner
Factor of Safety: 1.68



Liner detail from AECOM permit drawing P-17.

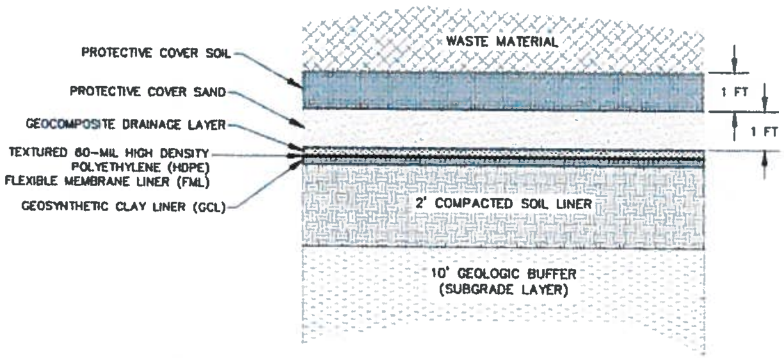
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

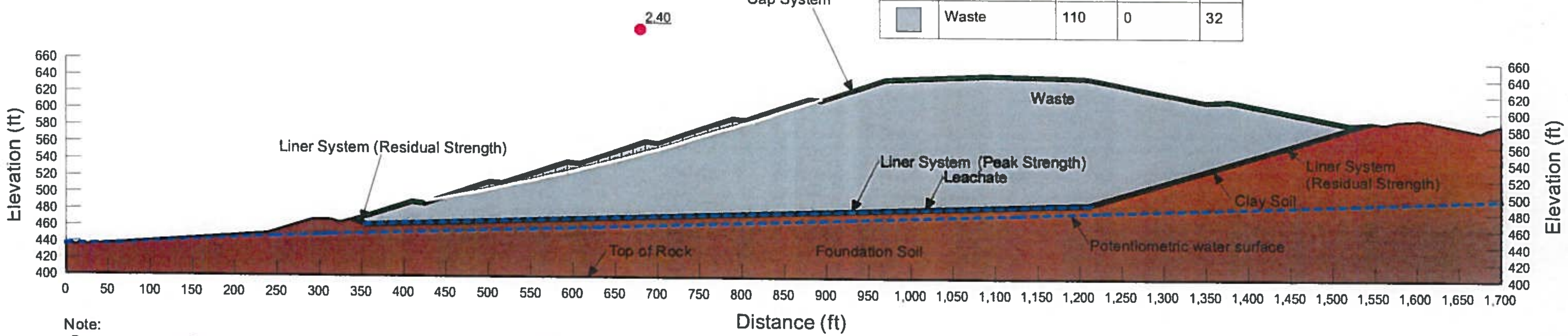
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Critical Section A-A'

Scenario F - Static
Slip surface: Shallow circular through waste
Factor of Safety: 2.40



Liner detail from AECOM permit drawing P-17.

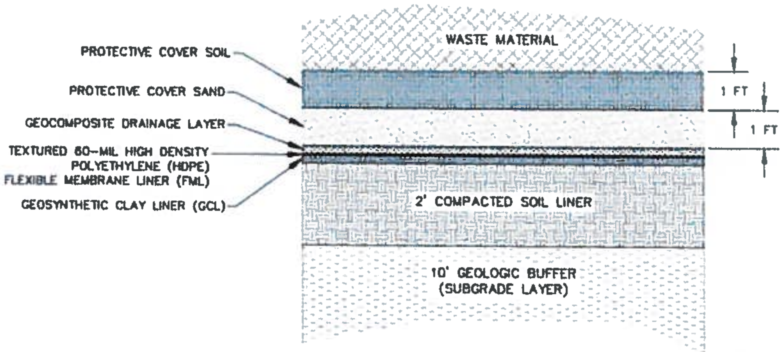
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

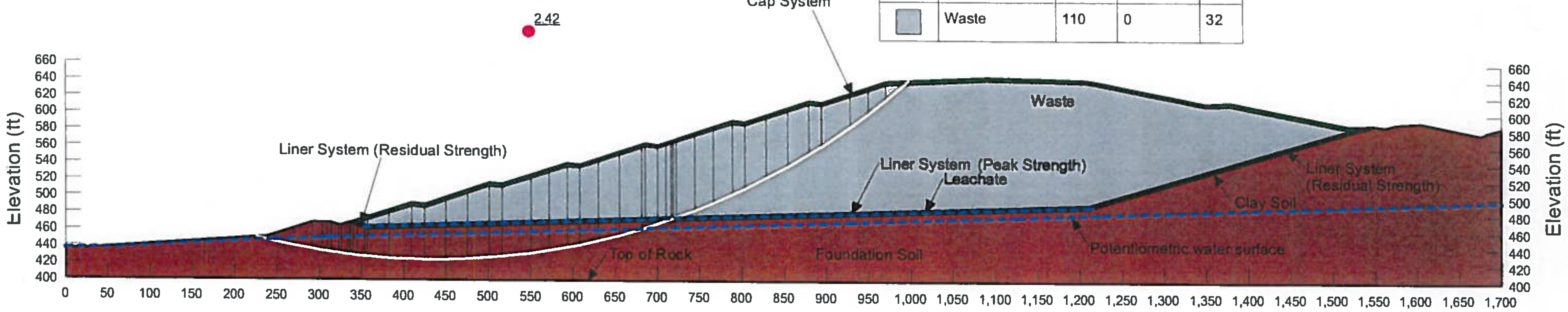
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Critical Section A-A'

Scenario G - Static
Slip surface: Large circular through foundation
Factor of Safety: 2.42



Liner detail from AECOM permit drawing P-17.

Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

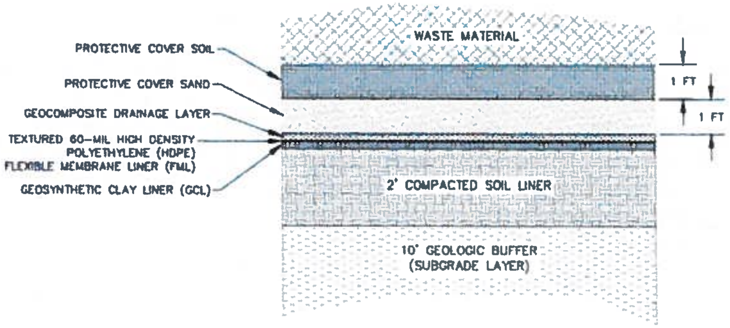
Attachment C Cross Section B-B'

Slope Stability Output

AECOM

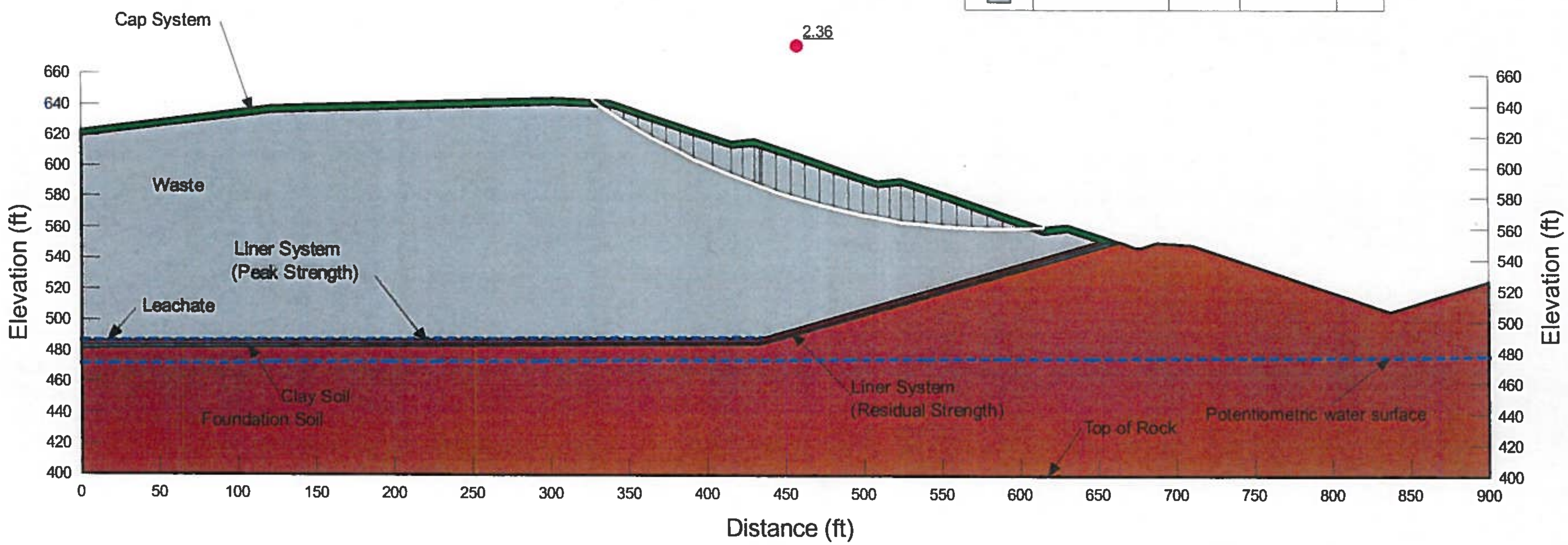
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section B-B'

Scenario A - Static
Slip surface: Large circular through waste
Factor of Safety: 2.36



Liner detail from AECOM permit drawing P-17.

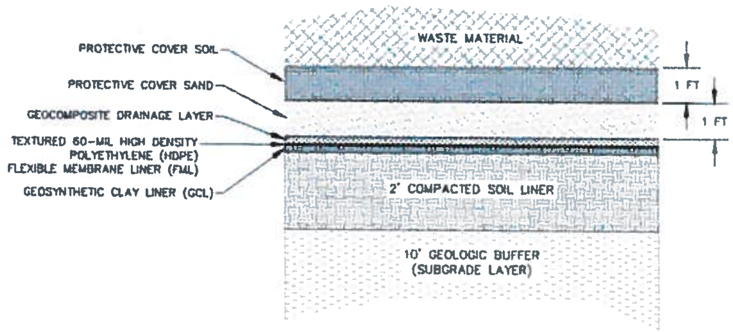
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

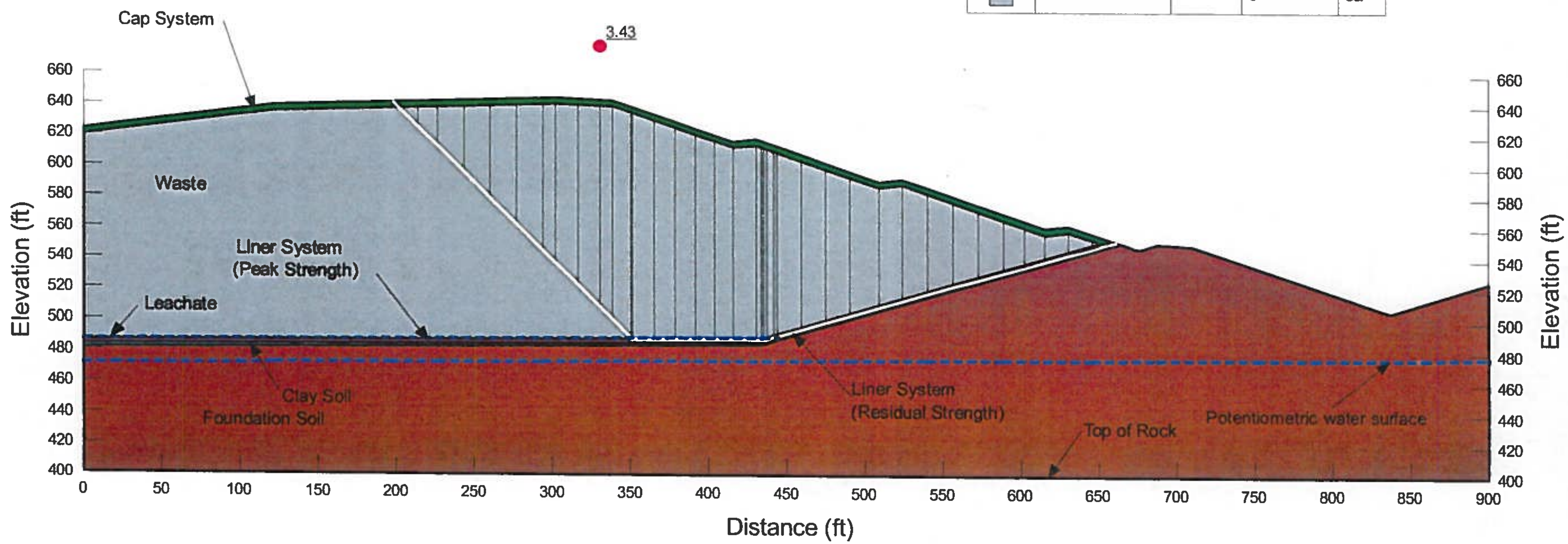
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section B-B'

Scenario B - Static
Slip surface: Translational along liner
Factor of Safety: 3.43



Liner detail from AECOM permit drawing P-17.

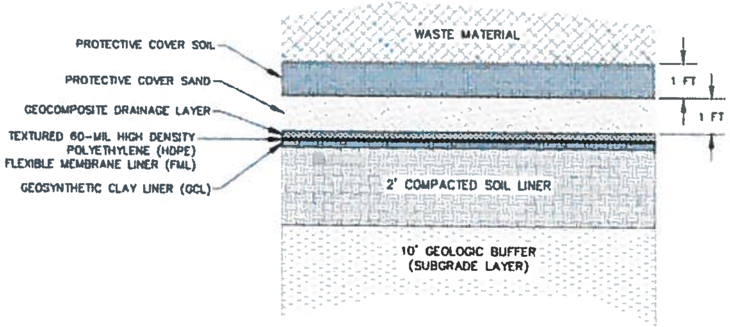
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

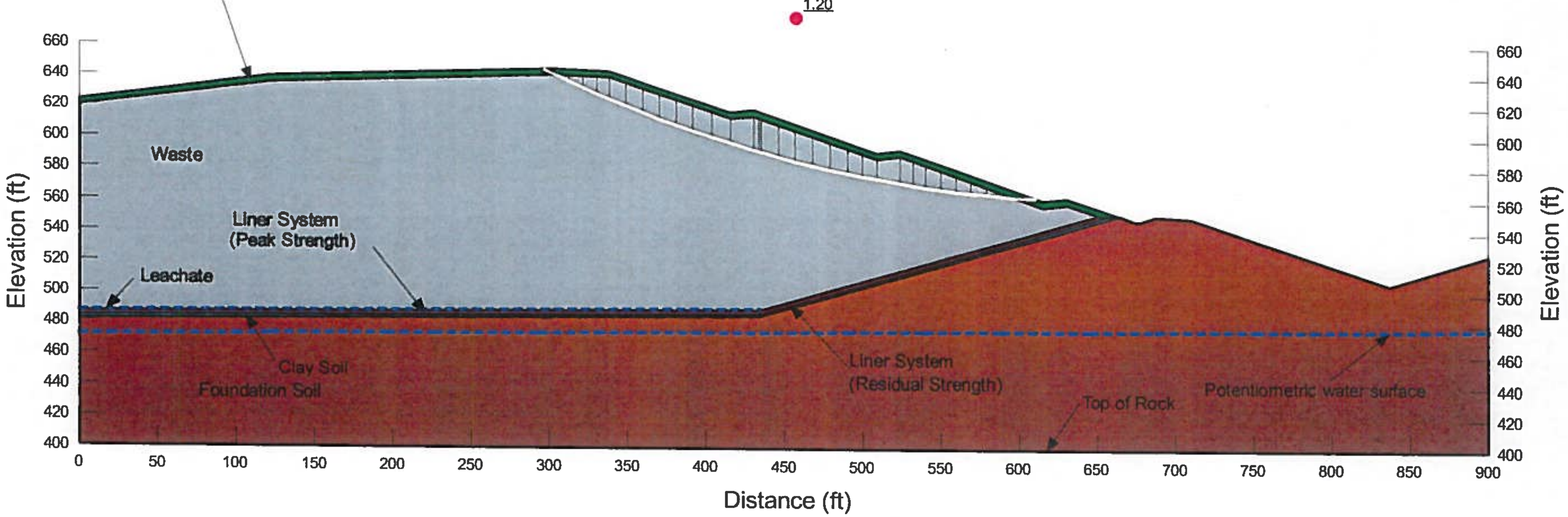
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section B-B'

Scenario C - Pseudostatic
Slip surface: Large circular through waste
Factor of Safety: 1.20
Horizontal Seismic Coefficient: 0.266g



Liner detail from AECOM permit drawing P-17.

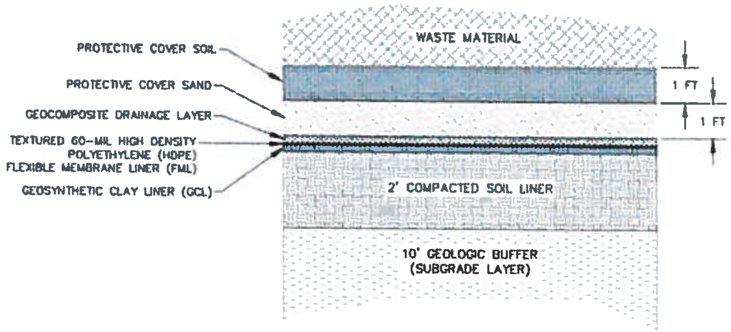
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

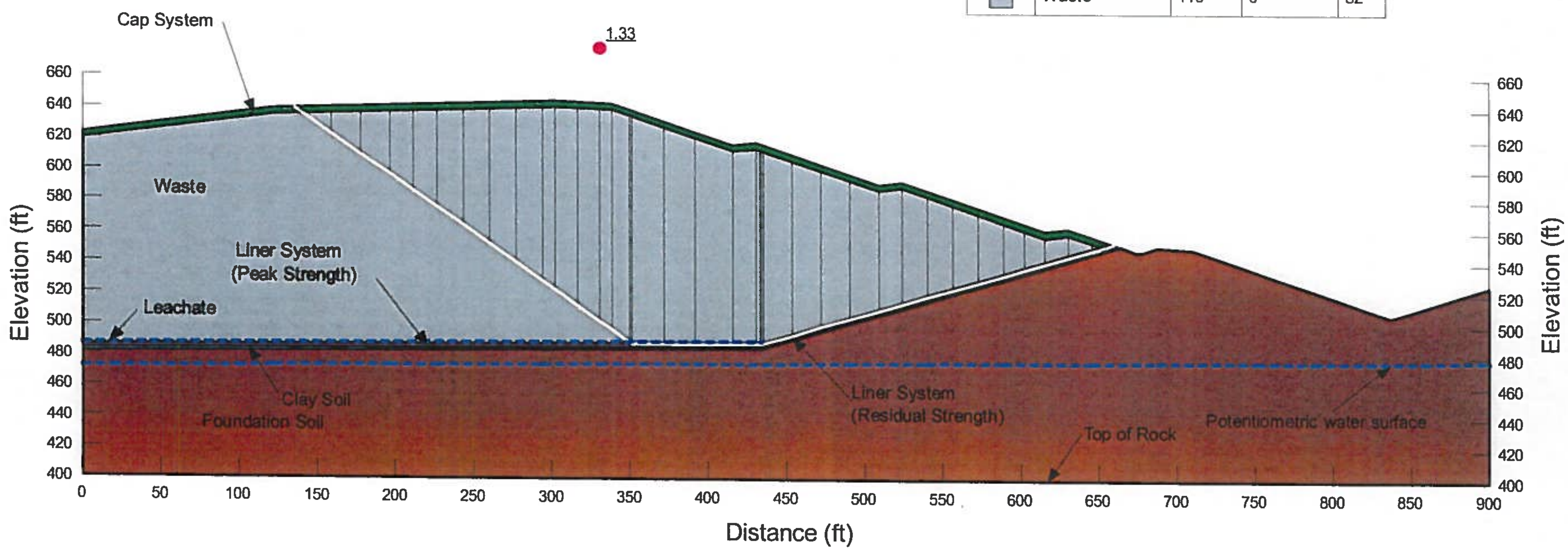
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section B-B'

Scenario D - Pseudostatic
Slip surface: Translational along liner
Factor of Safety: 1.33
Horizontal Seismic Coefficient: 0.266g



Liner detail from AECOM permit drawing P-17.

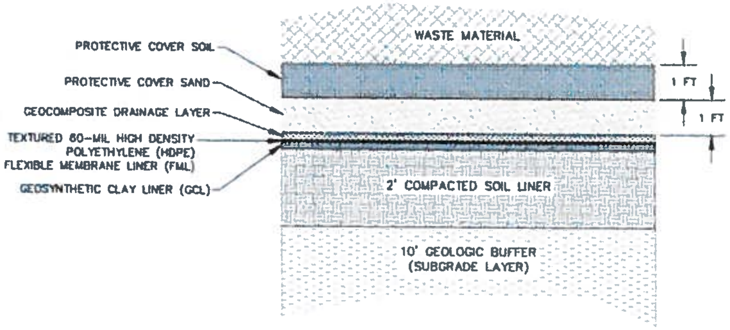
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

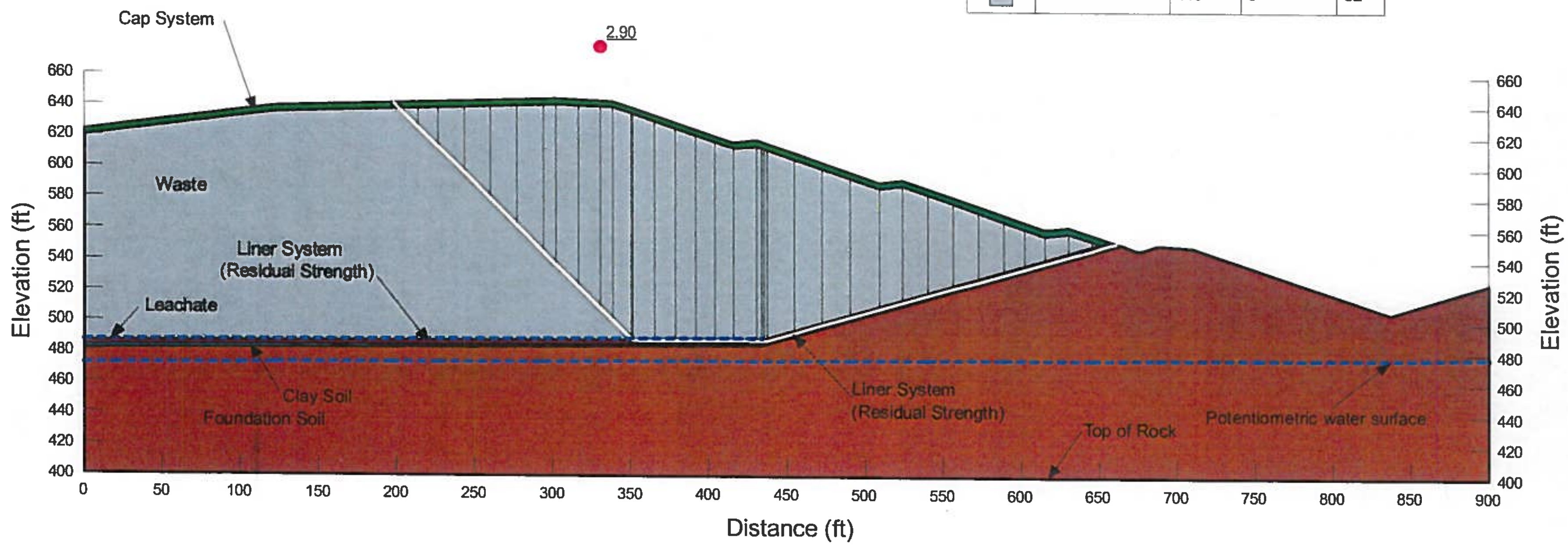
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section B-B'

Scenario E - Static
Slip surface: Translational along residual liner
Factor of Safety: 2.90



Liner detail from AECOM permit drawing P-17.

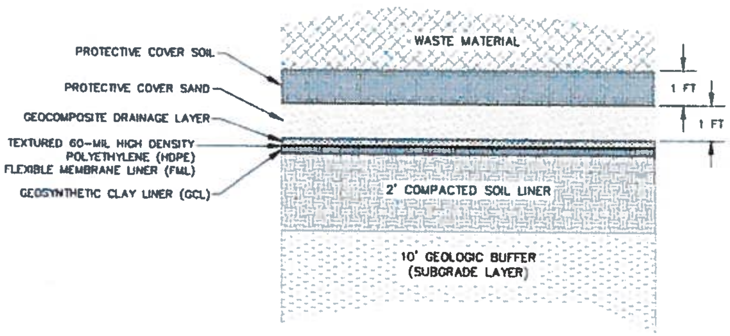
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

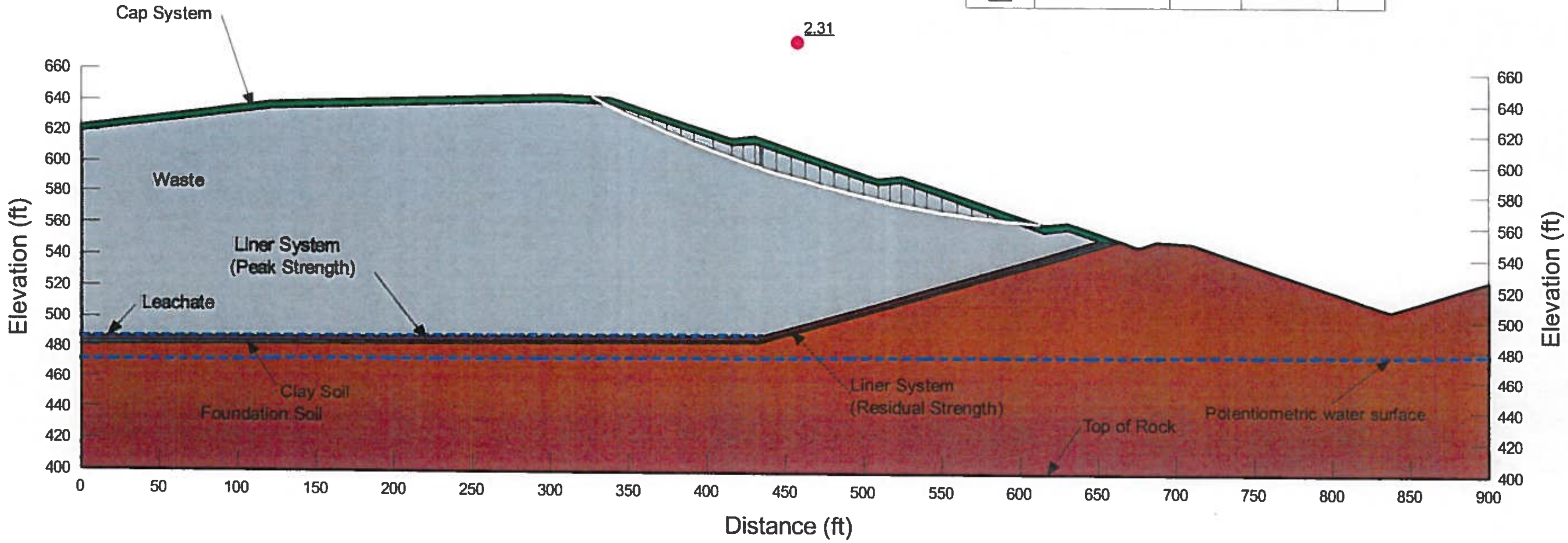
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section B-B'

Scenario F - Static
Slip surface: Shallow circular through waste
Factor of Safety: 2.31



Liner detail from AECOM permit drawing P-17.

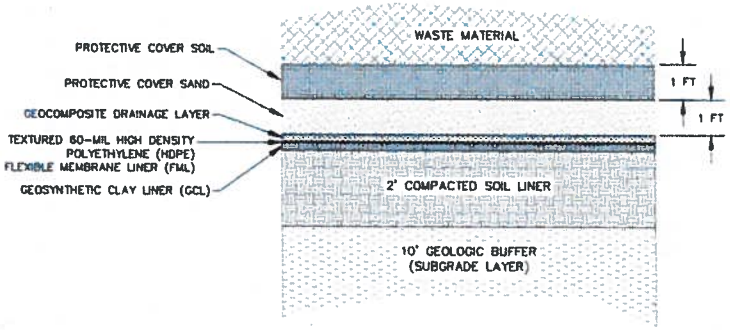
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

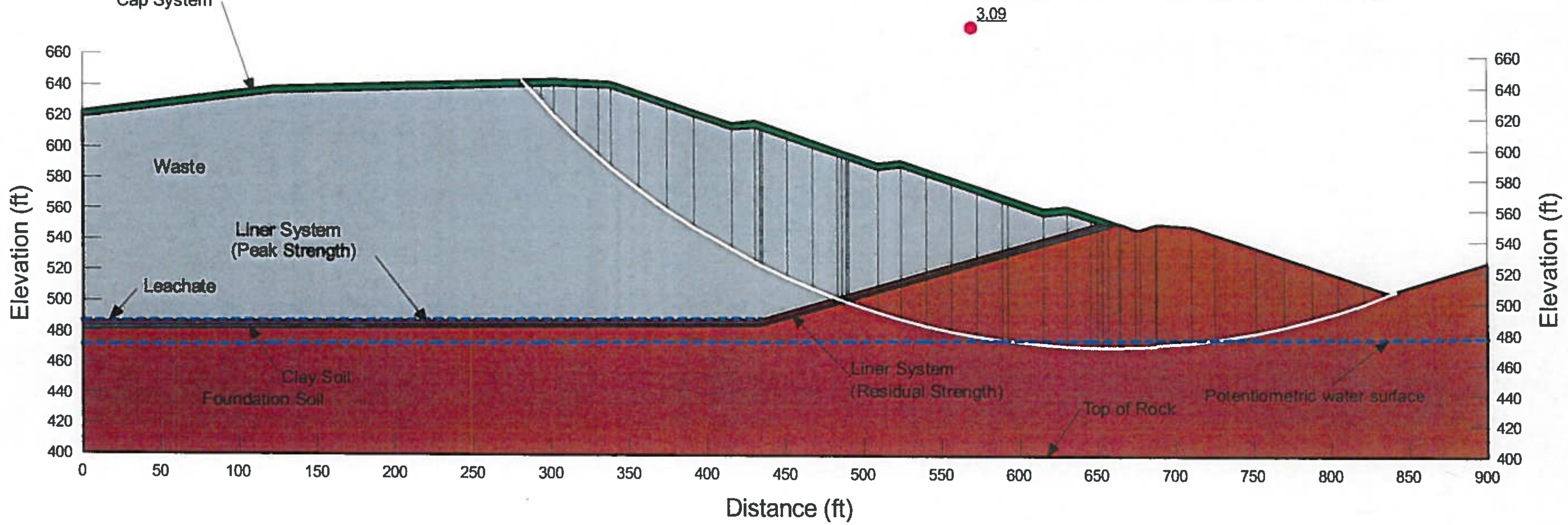
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section B-B'

Scenario G - Static
Slip surface: Large circular through foundation
Factor of Safety: 3.09



Liner detail from AECOM permit drawing P-17.

Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



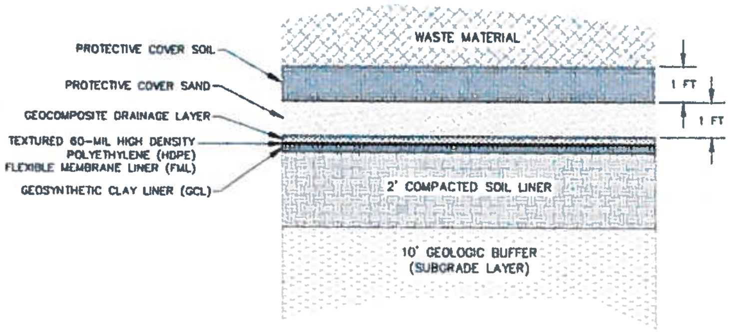
Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

Attachment D Cross Section C-C'

Slope Stability Output

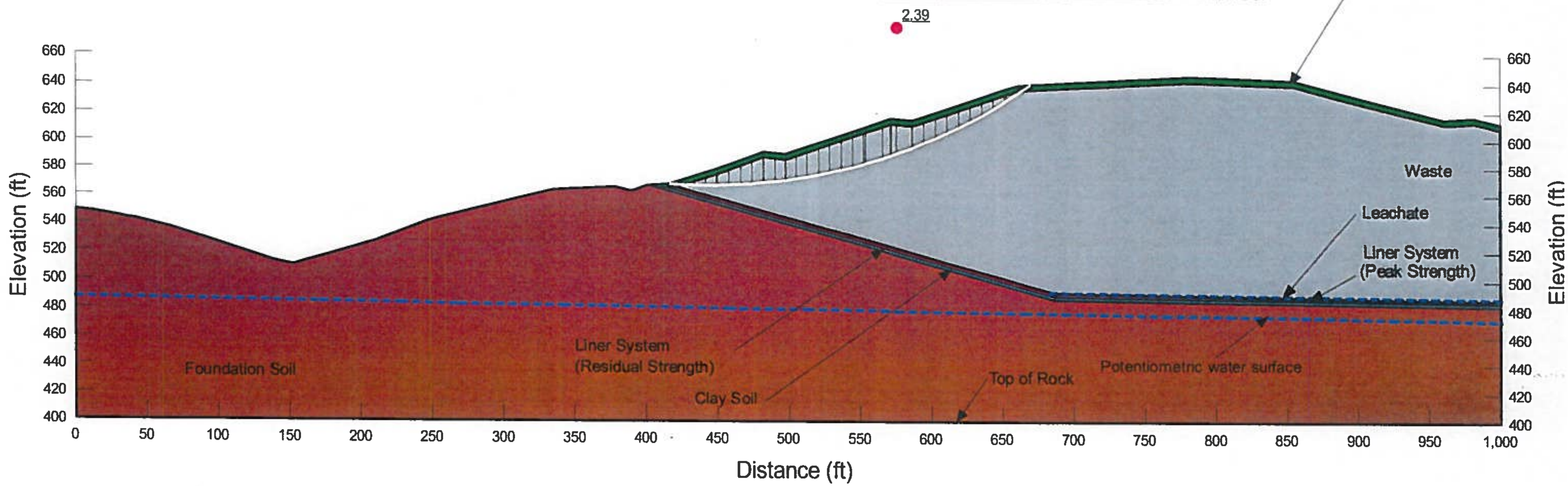
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section C-C'

Scenario A - Static
Slip surface: Large circular through waste
Factor of Safety: 2.39



Liner detail from AECOM permit drawing P-17.

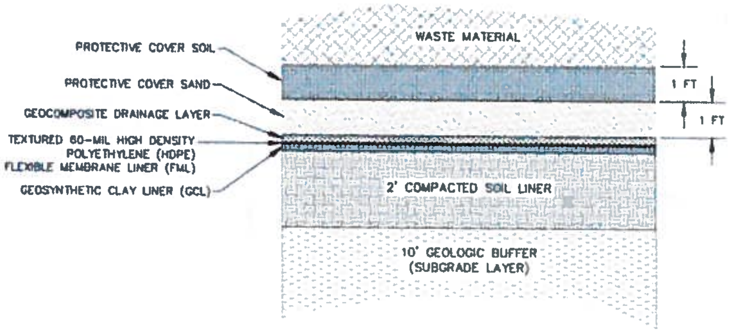
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

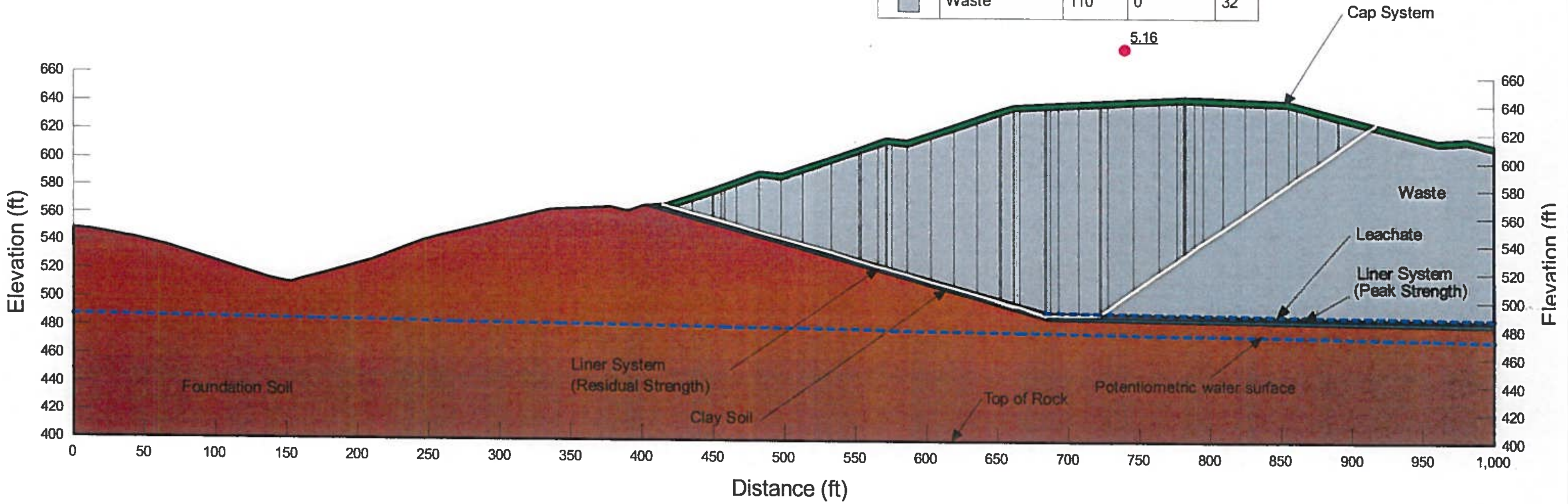
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section C-C'

Scenario B - Static
Slip surface: Translational along liner
Factor of Safety: 5.16



Liner detail from AECOM permit drawing P-17.

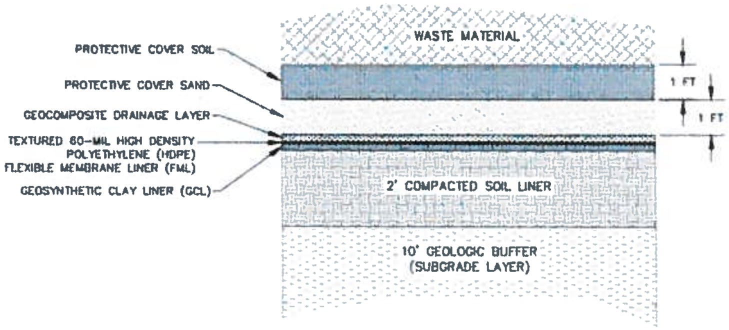
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

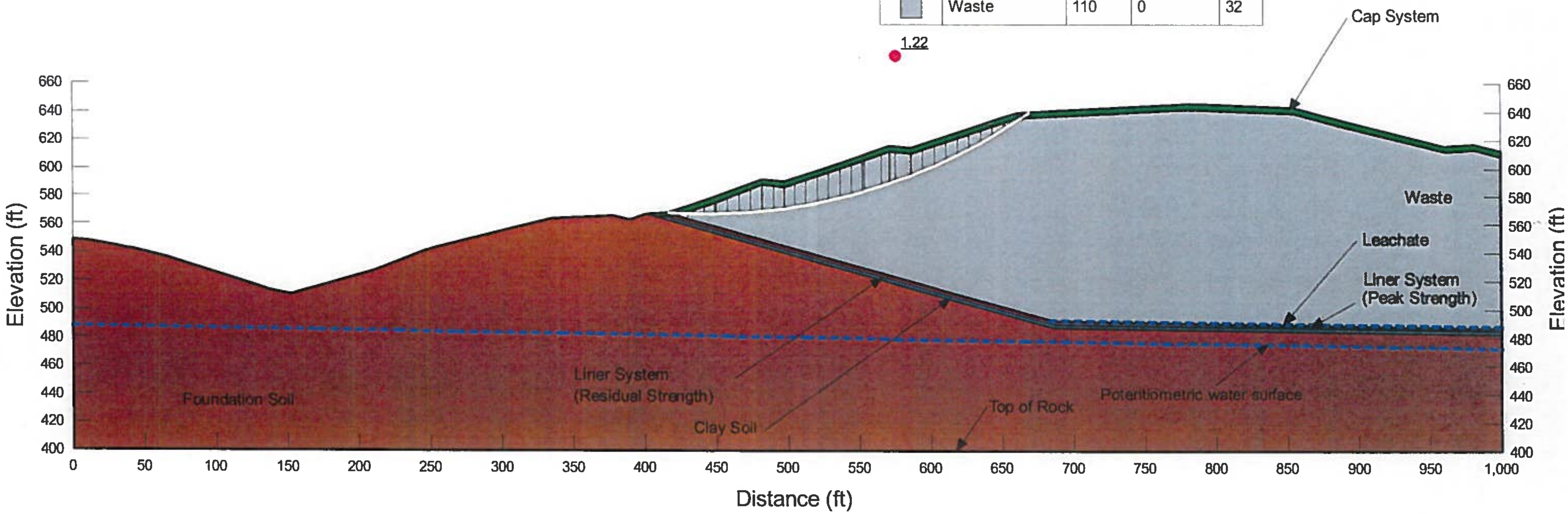
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section C-C'

Scenario C - Pseudostatic
Slip surface: Large circular through waste
Factor of Safety: 1.22
Horizontal Seismic Coefficient: 0.266g



Liner detail from AECOM permit drawing P-17.

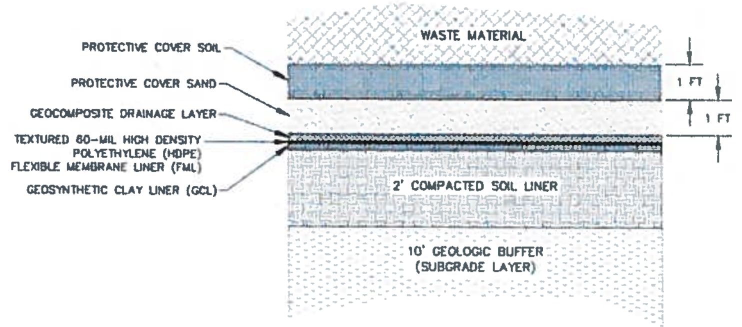
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

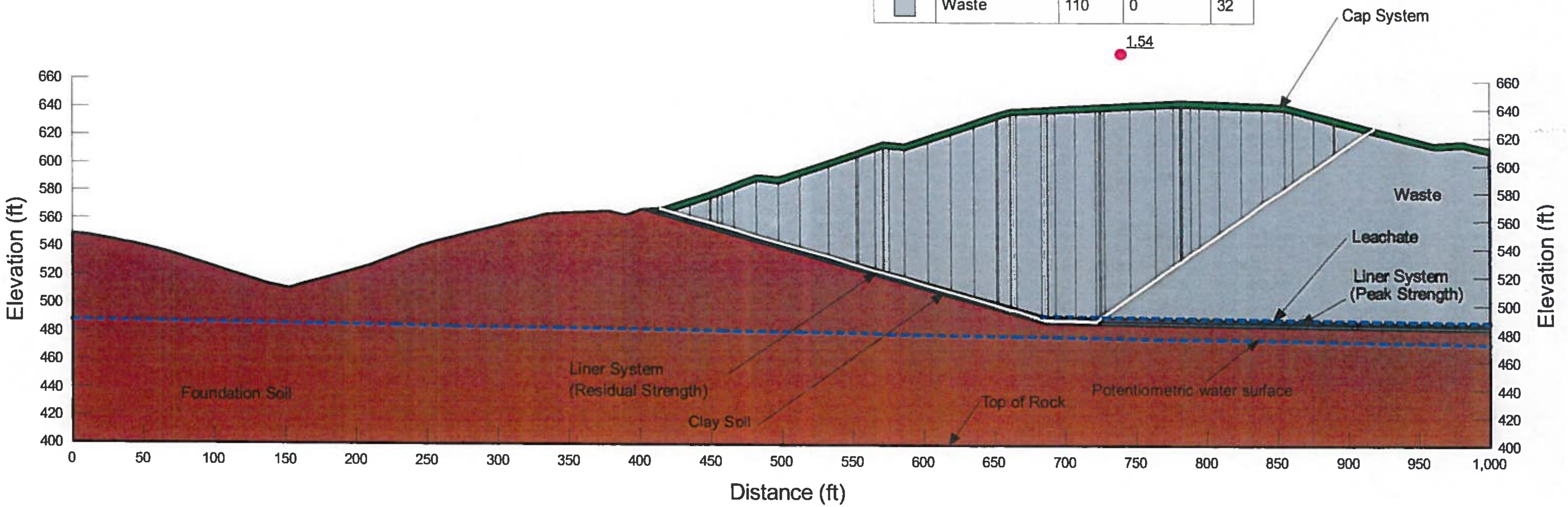
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section C-C'

Scenario D - Pseudostatic
Slip surface: Translational along liner
Factor of Safety: 1.54
Horizontal Seismic Coefficient: 0.266g



Liner detail from AECOM permit drawing P-17.

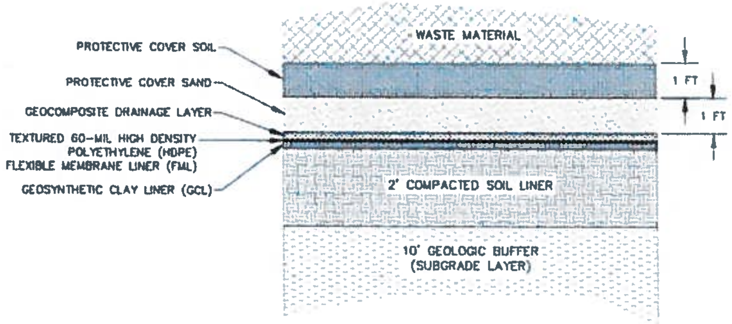
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

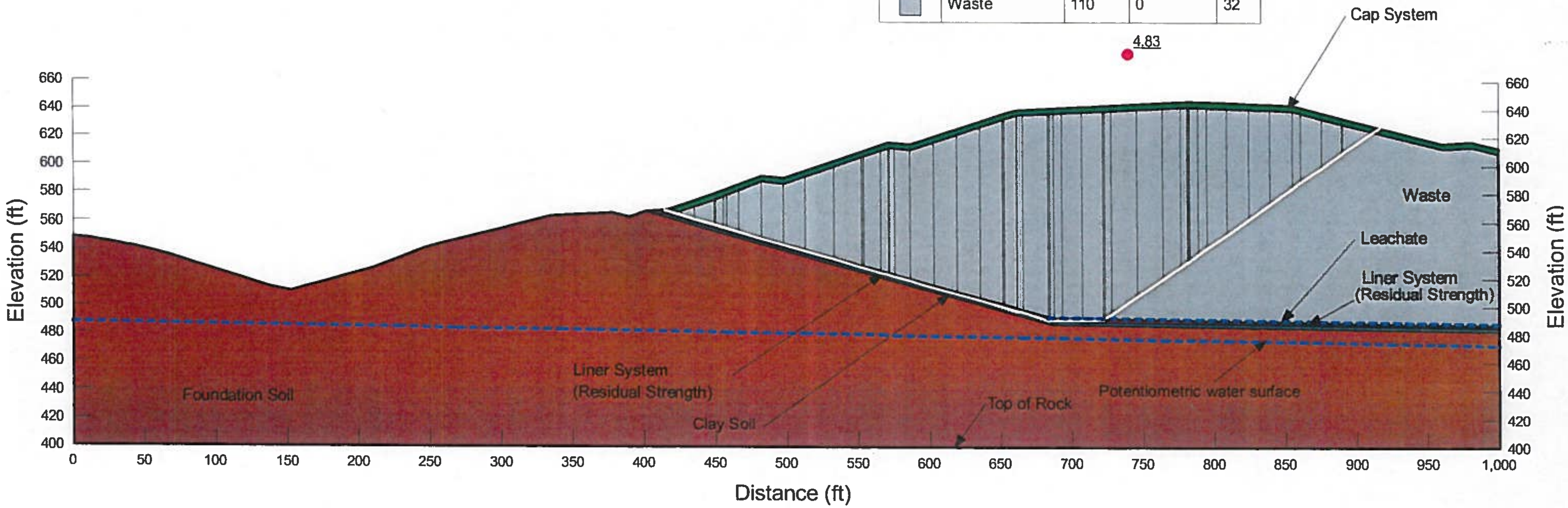
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section C-C'

Scenario E - Static
Slip surface: Translational along residual liner
Factor of Safety: 4.83



Liner detail from AECOM permit drawing P-17.

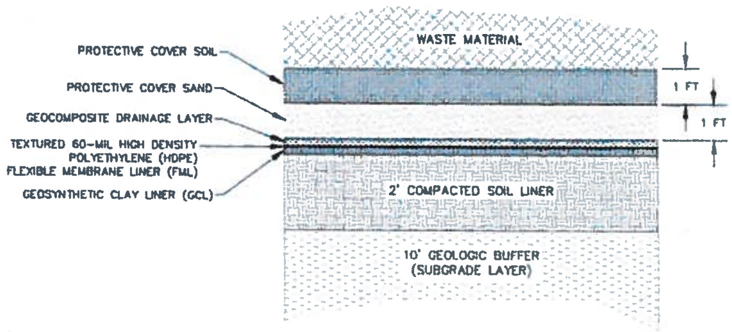
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

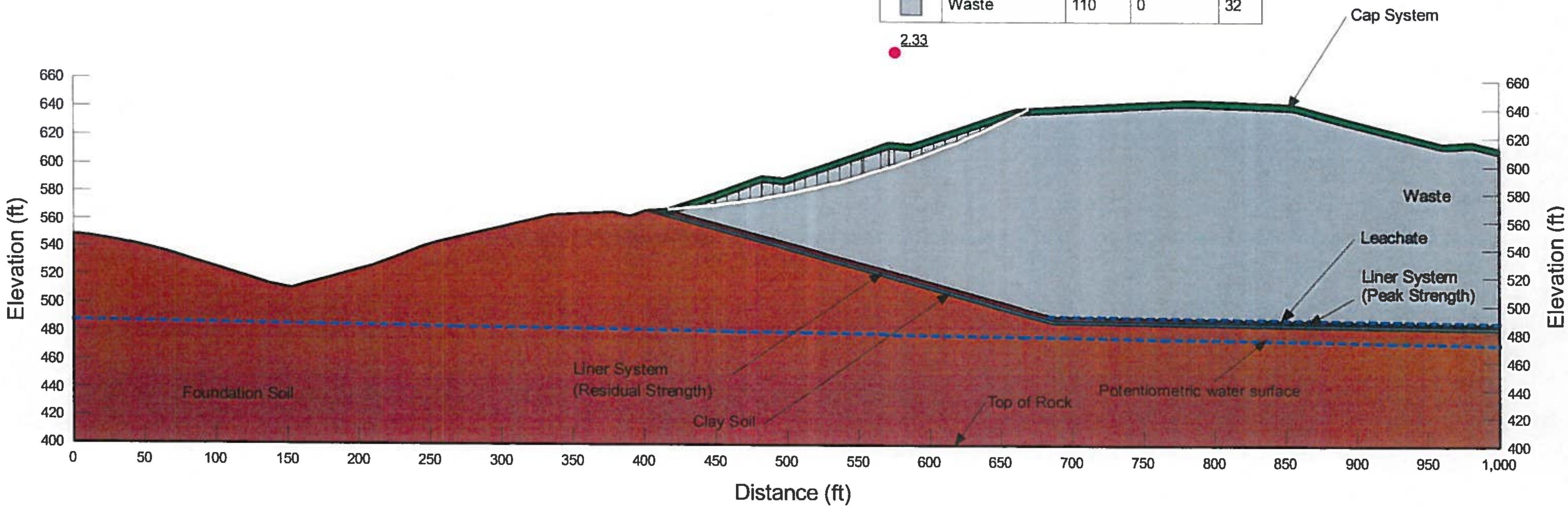
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section C-C'

Scenario F - Static
Slip surface: Shallow circular through waste
Factor of Safety: 2.33



Liner detail from AECOM permit drawing P-17.

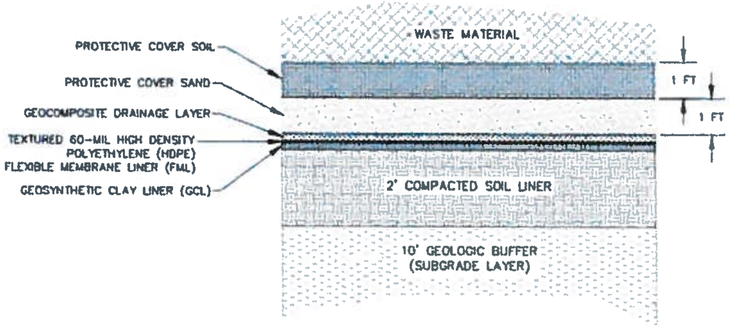
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

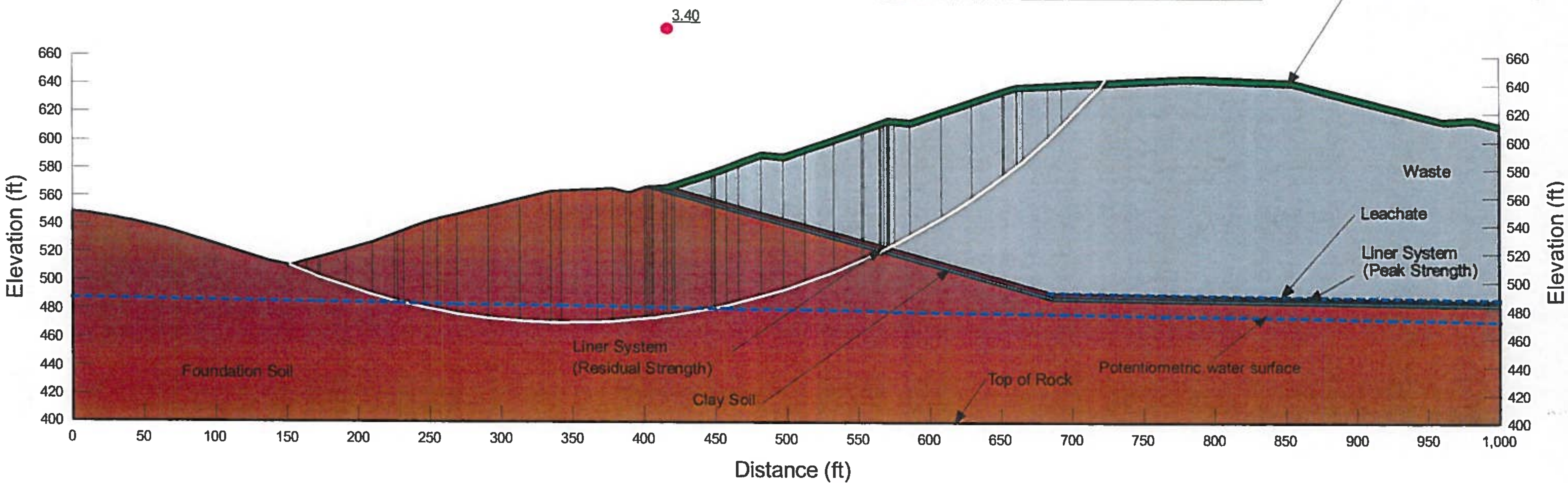
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section C-C'

Scenario G - Static
Slip surface: Large circular through foundation
Factor of Safety: 3.40



Liner detail from AECOM permit drawing P-17.

Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

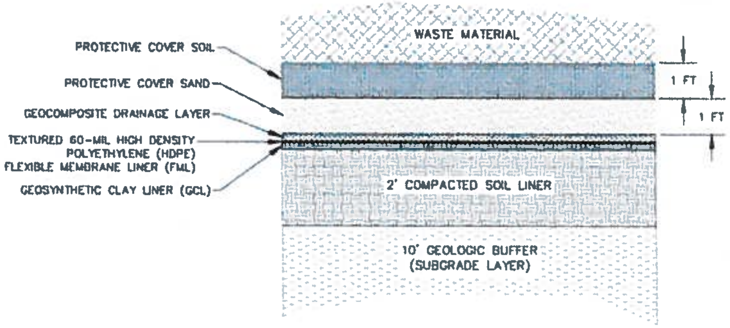
Attachment E Cross Section D-D'

Slope Stability Output

AECOM

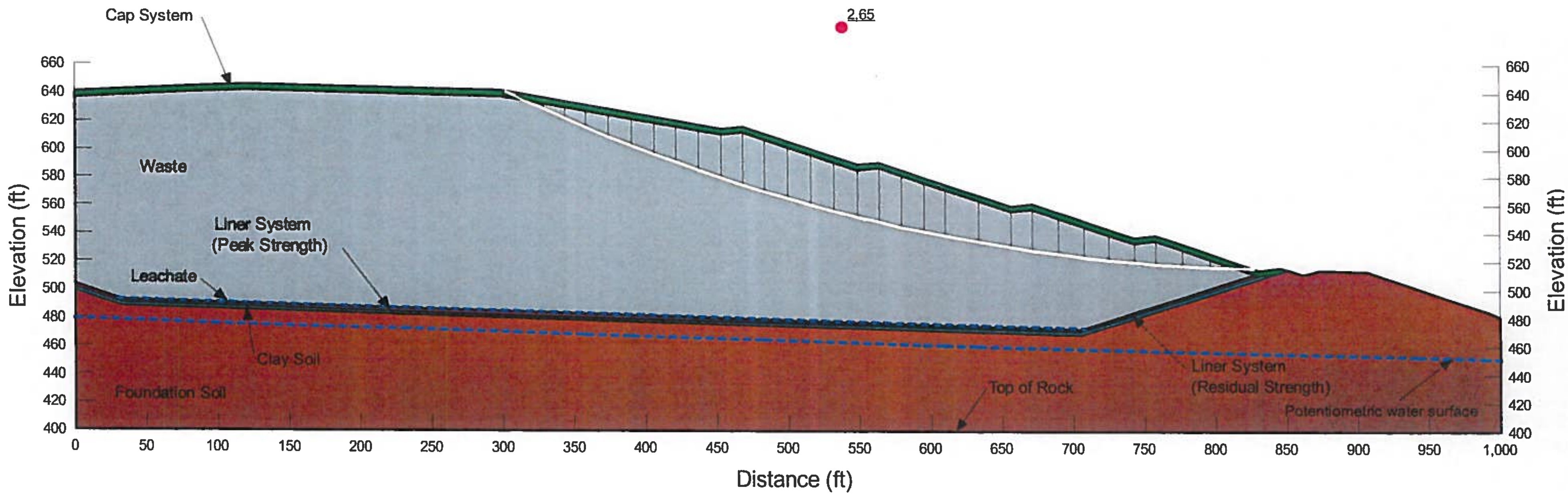
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section D-D'

Scenario A - Static
Slip surface: Large circular through waste
Factor of Safety: 2.65



Liner detail from AECOM permit drawing P-17.

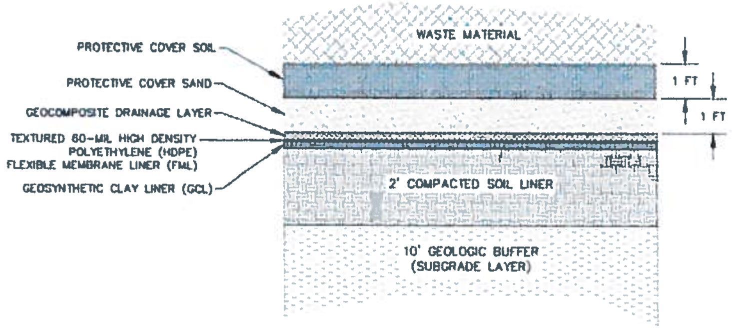
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

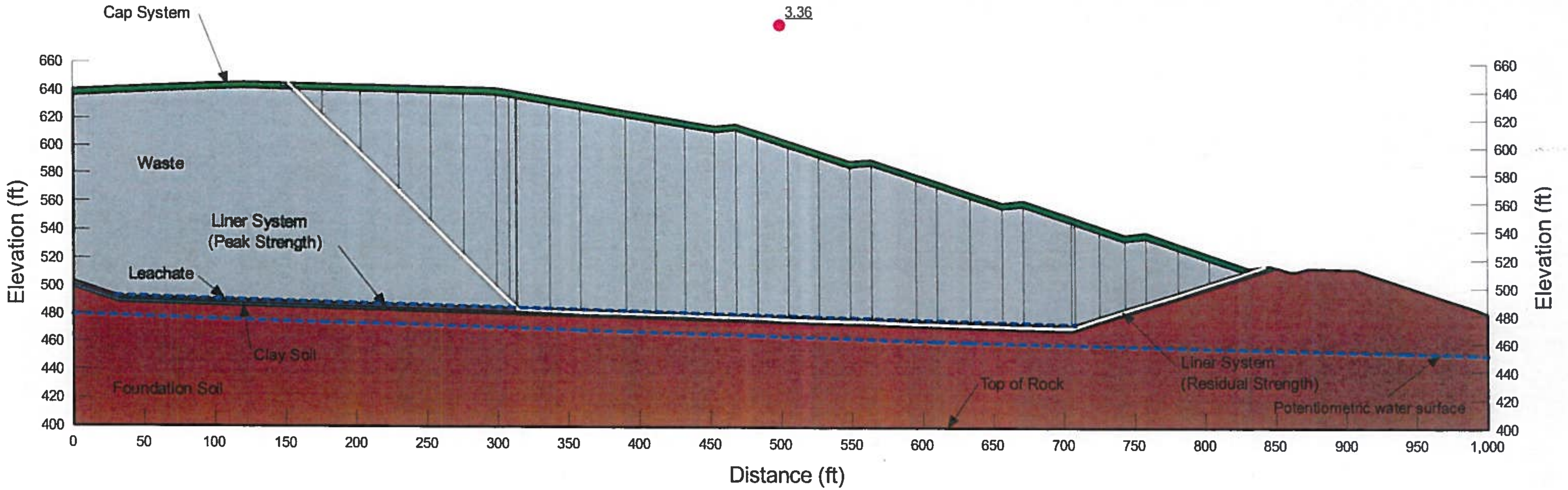
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section D-D'

Scenario B - Static
Slip surface: Translational along liner
Factor of Safety: 3.36



Liner detail from AECOM permit drawing P-17.

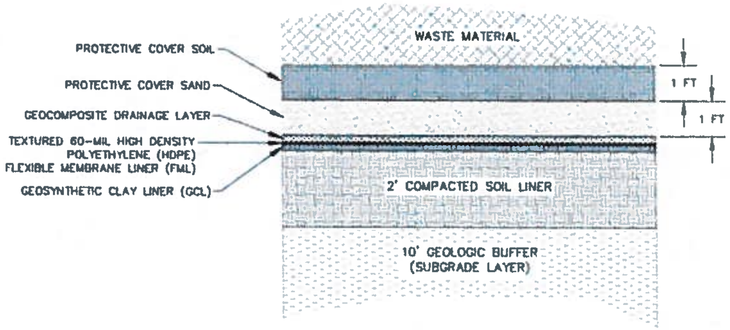
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

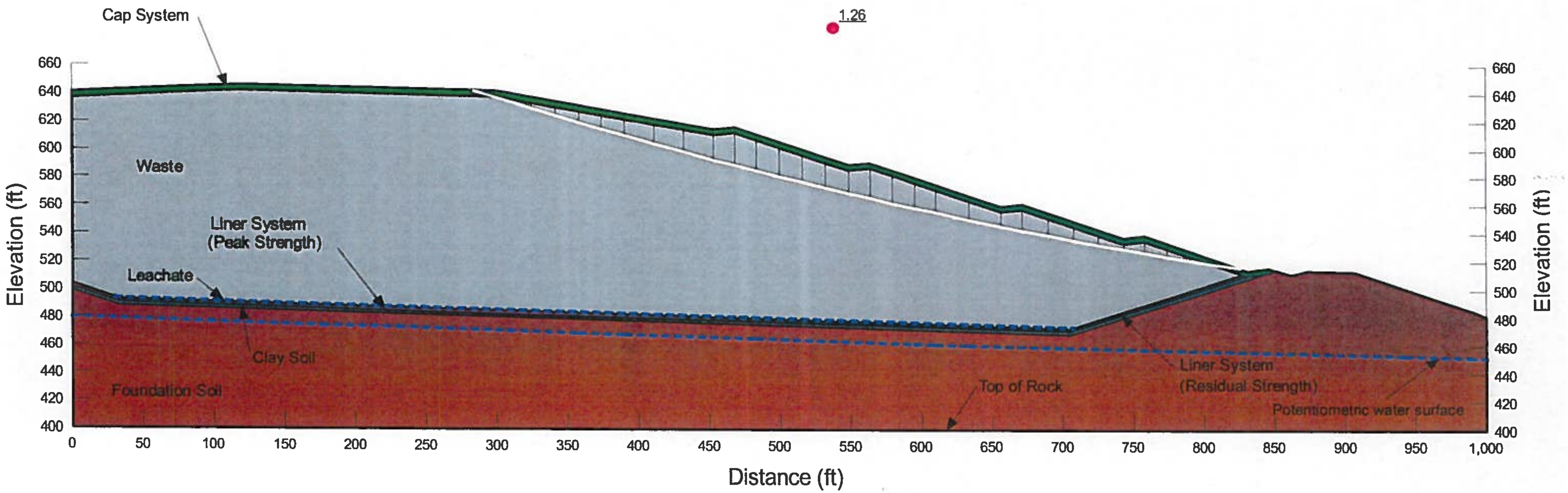
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section D-D'

Scenario C - Pseudostatic
Slip surface: Large circular through waste
Factor of Safety: 1.26
Horizontal Seismic Coefficient: 0.266g



Liner detail from AECOM permit drawing P-17.

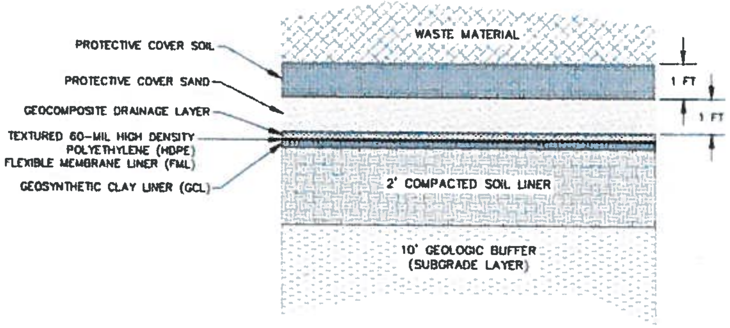
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

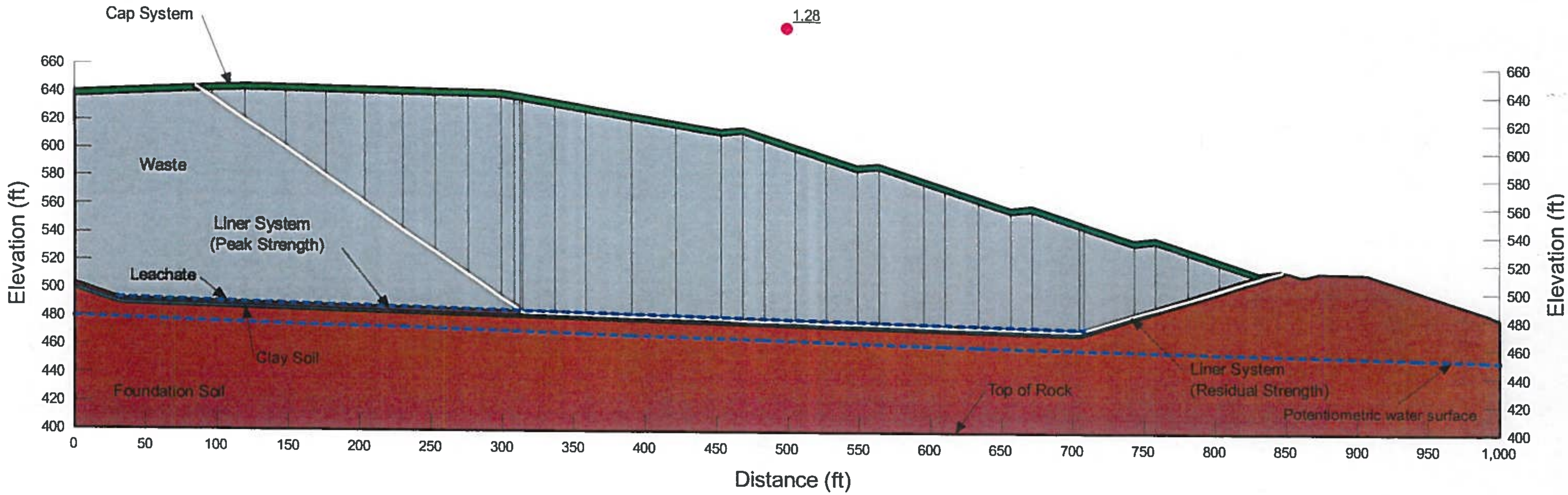
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section D-D'

Scenario D - Pseudostatic
Slip surface: Translational along liner
Factor of Safety: 1.28
Horizontal Seismic Coefficient: 0.266g



Liner detail from AECOM permit drawing P-17.

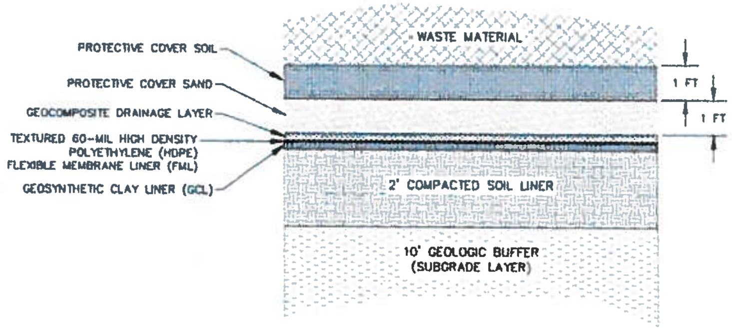
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

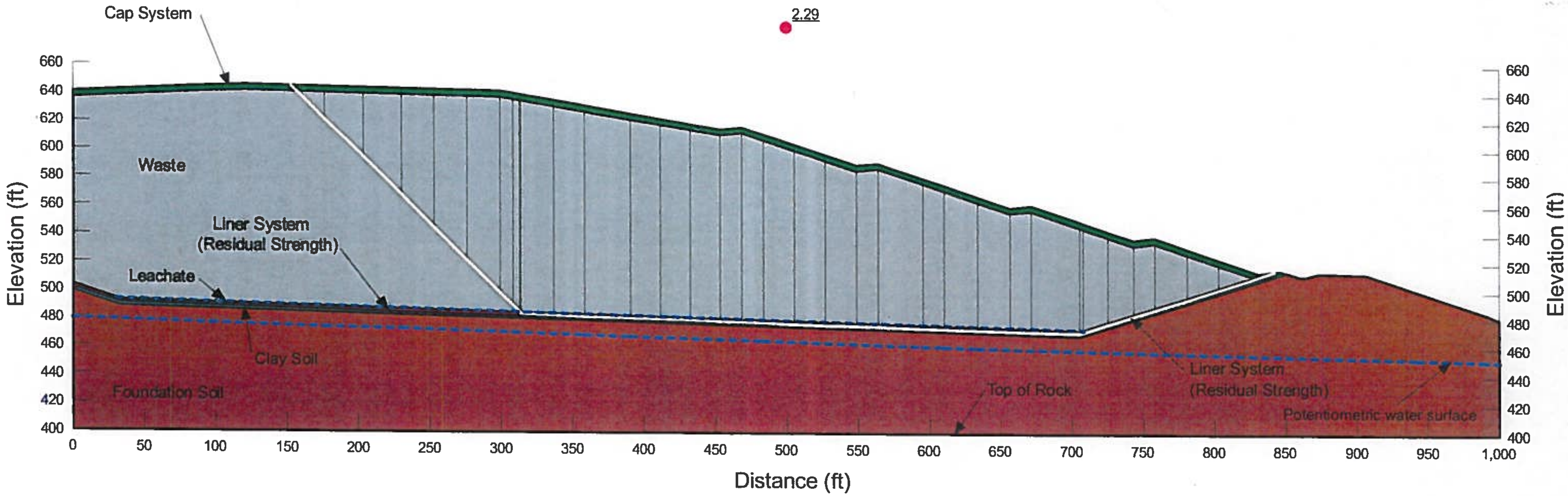
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section D-D'

Scenario E - Static
Slip surface: Translational along residual liner
Factor of Safety: 2.29



Liner detail from AECOM permit drawing P-17.

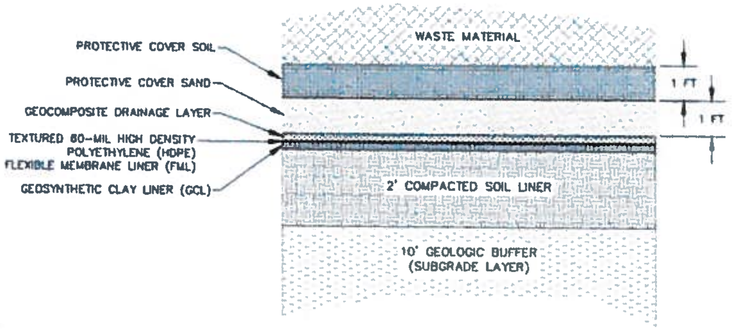
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

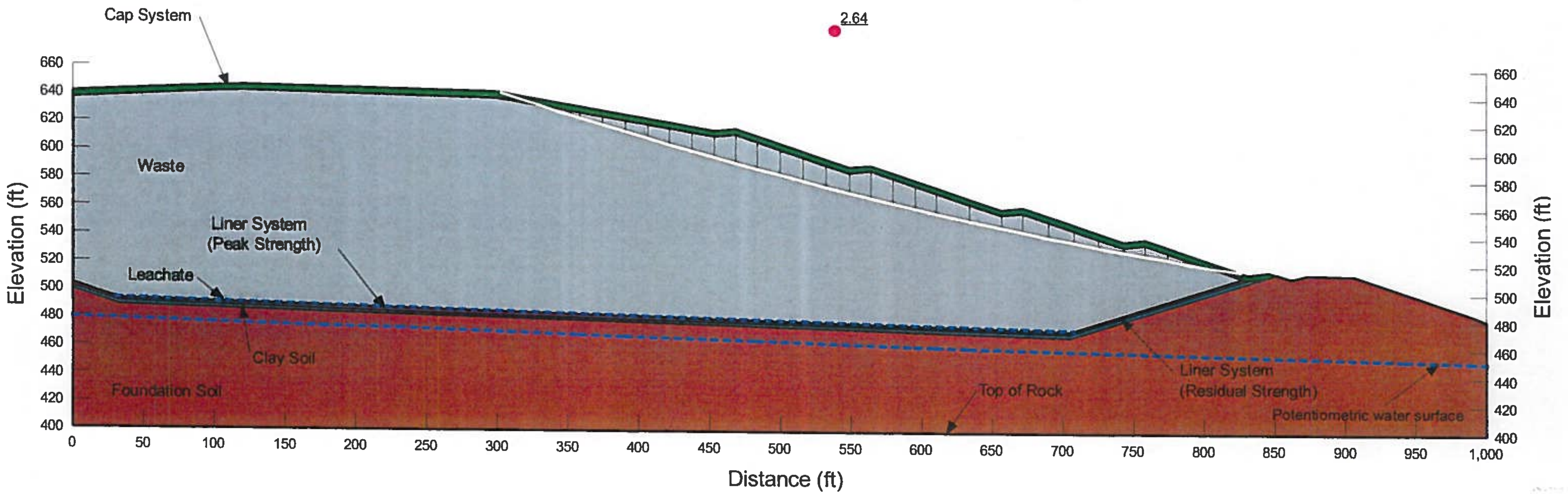
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section D-D'

Scenario F - Static
Slip surface: Shallow circular through waste
Factor of Safety: 2.64



Liner detail from AECOM permit drawing P-17.

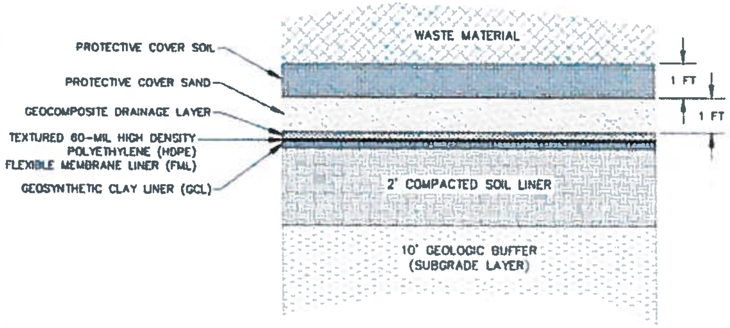
Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

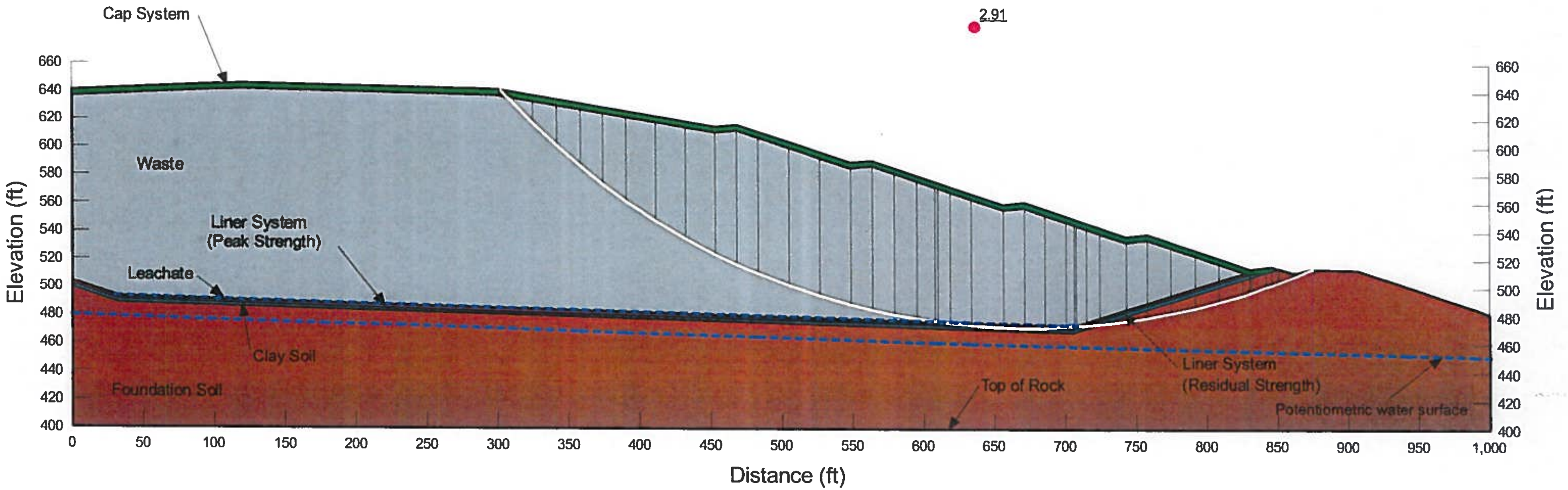
Scepter, Inc.
Class II Disposal Facility - East Phase
Waverly, TN
Cross Section D-D'

Scenario G - Static
Slip surface: Large circular through foundation
Factor of Safety: 2.91



Liner detail from AECOM permit drawing P-17.

Color	Name	Unit Weight	Cohesion'	Phi'
	Cap System	110	169	26
	Clay Soil	120	270	28
	Foundation Soil	120	0	32.5
	Liner System (Peak Strength)	110	0	26
	Liner System (Residual Strength)	110	0	15
	Waste	110	0	32



Note:
-Cap system and liner material properties based on published data in GRI Report #30.
-The interface with lowest shear strength (textured LLDPE on geotextile for cap system and textured HDPE on geocomposite for liner system) was used to model the overall cap system and liner system layers.

Attachment F

Critical Cross Section

AECOM

TABLE 1: SUMMARY OF STABILITY ANALYSES			
SLIP SURFACE	SLIP SURFACE MODE	SLIP SURFACE TYPE	SAFETY FACTOR
A	STATIC	ROTATIONAL	2.40
B	STATIC	TRANSLATIONAL	2.32
C	PSEUDOSTATIC (0.266g)	ROTATIONAL	1.20
D	PSEUDOSTATIC (0.266g)	TRANSLATIONAL	1.04
E	STATIC–RESIDUAL STRENGTH	TRANSLATIONAL	1.53
F	STATIC–SHALLOW SLOUGHING IN WASTE	ROTATIONAL	2.40
G	STATIC–GLOBAL	ROTATIONAL	2.40

TABLE 2: MATERIAL PROPERTIES			
MATERIAL	UNIT WEIGHT, pcf	COHESION, psf	FRICTION ANGLE, deg
CAP SYSTEM	110	169	26
WASTE	110	0	32
LINER SYSTEM (PEAK STRENGTH)*	65	167	23
LINER SYSTEM (RESIDUAL STRENGTH)*	65	0	13
CLAY SOIL**	120	270	28
FOUNDATION SOIL***	120	0	32.5

* OBTAINED FROM PUBLISHED DATA IN GRI REPORT #30, SEE TABLE 3. LINER SYSTEM MODELED TO INCLUDE PROTECTIVE COVER SOIL AND PROTECTIVE COVER SAND WITH GEOCOMPOSITE/FLEXIBLE MEMBRANE LINER.
**CLAY SOIL DATA FROM NAVFAC DM7-02, TABLE 1-"TYPICAL PROPERTIES OF COMPACTED SOILS."
***FOUNDATION SOIL DATA BASED ON HYDROGEOLOGICAL EXPLORATION PERFORMED BY AECOM DATED APRIL 11, 2016.

TABLE 3: SUMMARY OF INTERFACE SHEAR STRENGTHS FROM GRI REPORT #30											
INTERFACE 1	INTERFACE 2	PEAK STRENGTH					RESIDUAL STRENGTH				
		δ (deg)	Ca (kPa)	Ca (psf)	POINTS	R^2	δ (deg)	Ca (kPa)	Ca (psf)	POINTS	R^2
HDPE–T	GCL	23	8.0	167	180	0.95	13	0	0	157	0.90
LLDPE–T	NW–NP GT	26	8.1	169	9	1.00	17	9.5	198	9	0.96

NOTE: THE CRITICAL INTERFACE IS TEXTURED HDPE ON GEOSYNTHETIC CLAY LINER (GCL) FOR THE LINER SYSTEM AND TEXTURED LLDPE ON GEOTEXTILE FOR THE CAP SYSTEM.

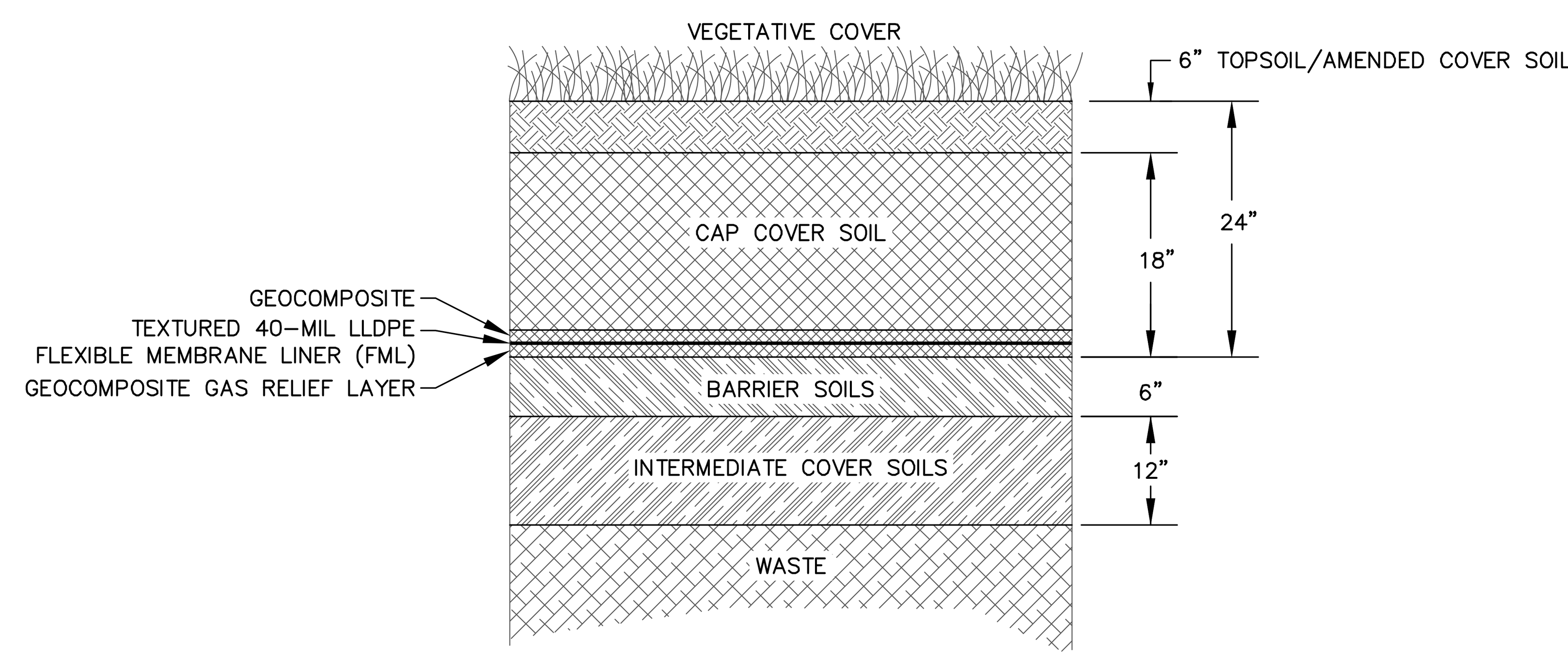
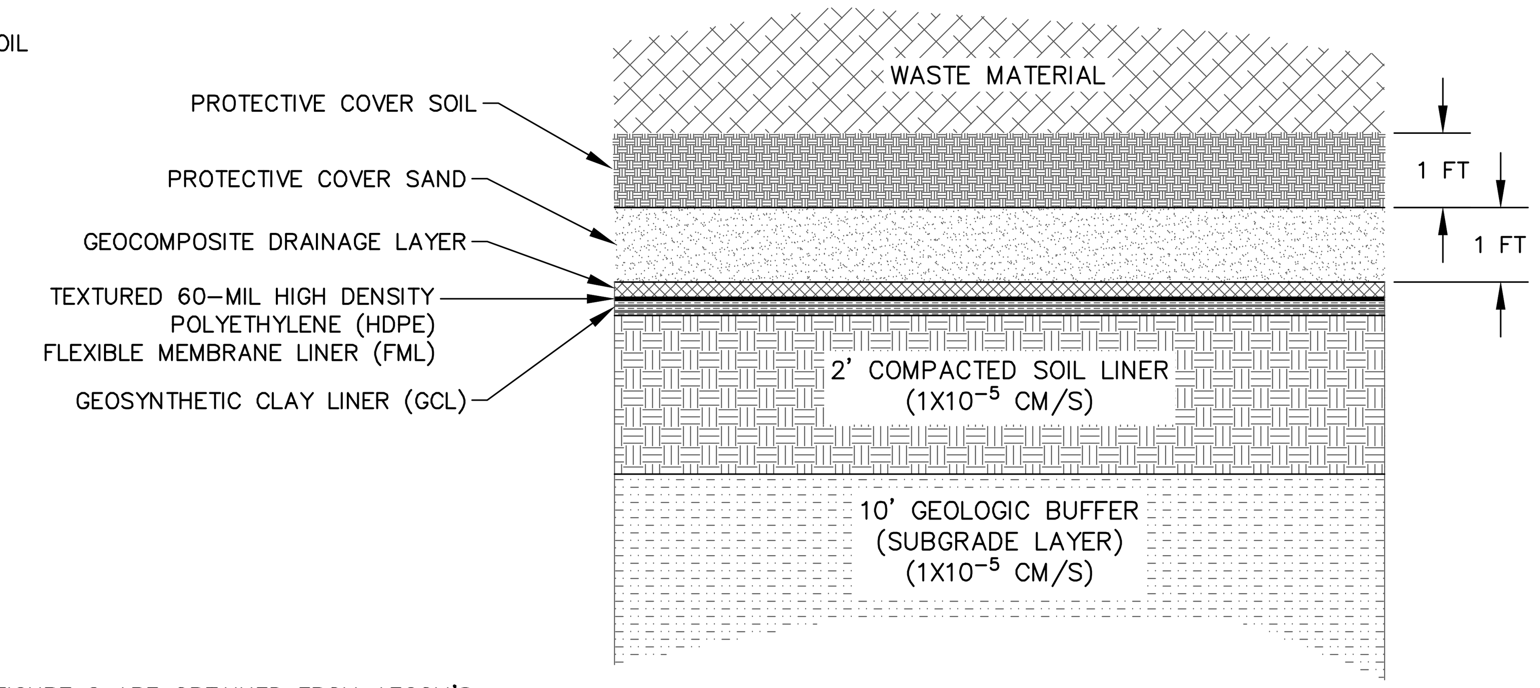
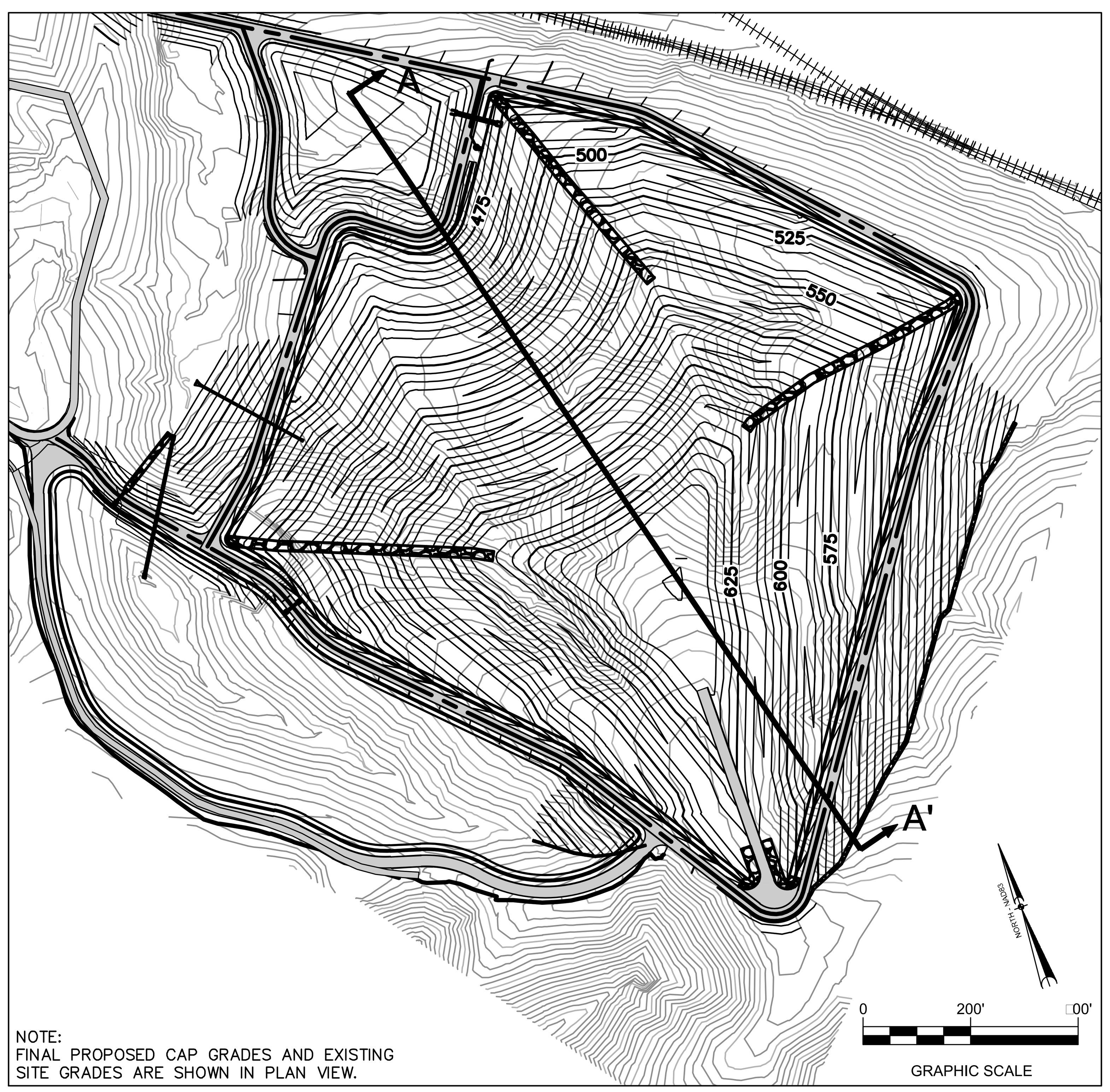


FIGURE 1: CAP SYSTEM



NOTE: FIGURE 1 AND FIGURE 2 ARE OBTAINED FROM AECOM'S PERMIT DRAWING P-17 AND ARE NOT DRAWN TO SCALE.

FIGURE 2: LINER SYSTEM



NOTE:
FINAL PROPOSED CAP GRADES AND EXISTING SITE GRADES ARE SHOWN IN PLAN VIEW.

FIGURE 3: PLAN VIEW

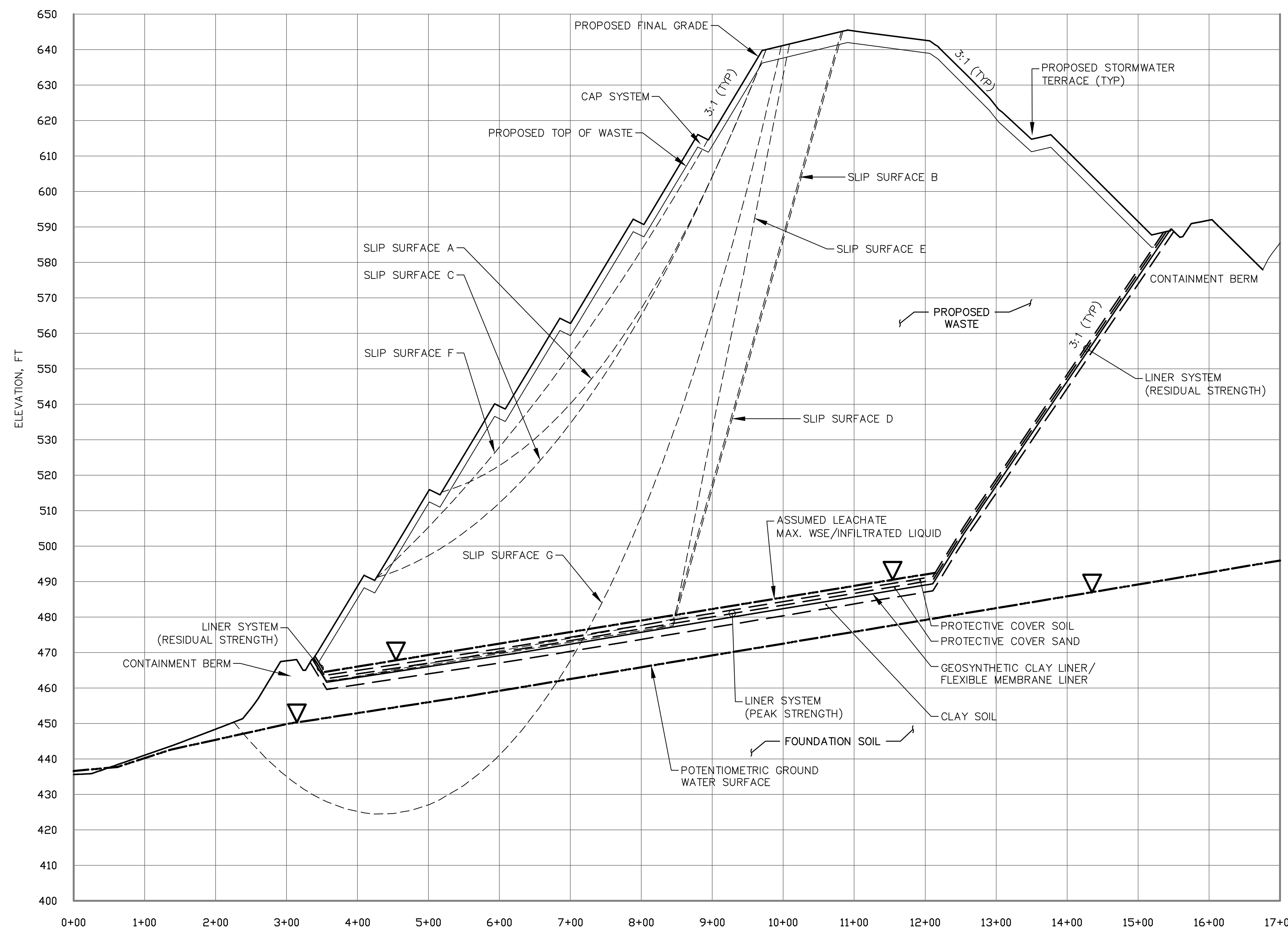


FIGURE 4: CRITICAL CROSS SECTION A-A' (NORTH)

DRAWN BY: SD	DATE 03/19/2018
CHECKED BY: MW	JOB NO.: 60398526
SCALE AS SHOWN	

CRITICAL CROSS SECTION FOR SLOPE STABILITY

PROPOSED EAST PHASE DISPOSAL FACILITY

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

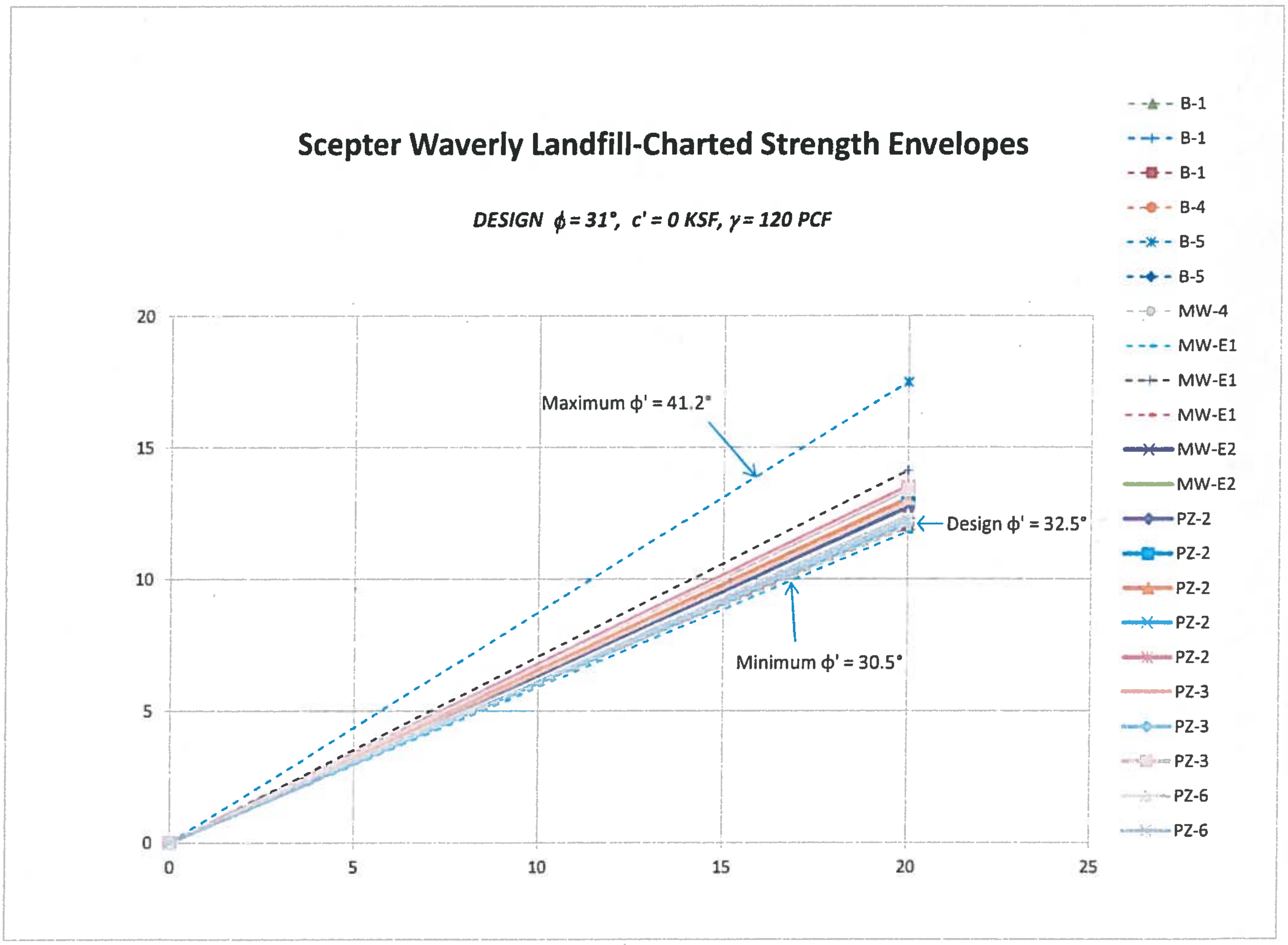
EXHIBIT 1

C:\Users\DouangvilayS\appdata\local\temp\AcPublish_7640\CRITICAL CROSS SECTION_03-19-18.dwg User:DouangvilayS Mar 19, 2018 - 11:52am

Attachment G

Charted Shear Strength Envelope for Foundation Soils

AECOM



Scepter Waverly Landfill-Charted Strength Envelopes
DESIGN ϕ = 31°, c' = 0 KSF, γ = 120 PCF
Maximum ϕ' = 41.2°
Design ϕ' = 32.5°
Minimum ϕ' = 30.5°
0
5
10
15
20
25
B-1
B-1
B-1
B-4
B-5
B-5
MW-4
MW-E1
MW-E1
MW-E1
MW-E2
MW-E2
PZ-2
PZ-2
PZ-2
PZ-2
PZ-2
PZ-3
PZ-3
PZ-3
PZ-6
PZ-6

Attachment H

GRI Report #30 Published Data

AECOM

Drexel
UNIVERSITY

Geosynthetic Research Institute
475 Kedron Avenue
Folsom, PA 19033-1208 USA
TEL (610) 522-8440
FAX (610) 522-8441

GRI GEI GII GSI GAI GCI

Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces

by

George R. Koerner, Ph.D., P.E.
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and

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GRI Report #30

June 14, 2005

Appendix Table 1. Summary of interface shear strengths.

	Interface 1*	Interface 2*	Peak Strength					Residual Strength				
			Fig. No.	δ (deg)	Ca (kPa)	Points	R^2	Fig. No.	δ (deg)	Ca (kPa)	Points	R^2
	HDPE-S	Granular Soil	1a	21	0	162	0.93	1b	17	0	128	0.92
	HDPE-S	Cohesive Soil										
		Saturated	1c	11	7	79	0.94	1d	11	0	59	0.95
		Unsaturated	1c	22	0	44	0.93	1d	18	0	32	0.93
	HDPE-S	NW-NP GT	1e	11	0	149	0.93	1f	9	0	82	0.96
	HDPE-S	Geonet	1g	11	0	196	0.90	1h	9	0	118	0.93
	HDPE-S	Geocomposite	1i	15	0	36	0.97	1j	12	0	30	0.93
	HDPE-T	Granular Soil	2a	34	0	251	0.98	2b	31	0	239	0.96
	HDPE-T	Cohesive Soil										
		Saturated	2c	18	10	167	0.93	2d	16	0	150	0.90
		Unsaturated	2c	19	23	62	0.91	2d	22	0	35	0.93
	HDPE-T	NW-NP GT	2e	25	8	254	0.96	2f	17	0	217	0.95
	HDPE-T	Geonet	2g	13	0	31	0.99	2h	10	0	27	0.99
LINER	HDPE-T	Geocomposite	2i	26	0	168	0.95	2j	15	0	164	0.94
	LLDPE-S	Granular Soil	3a	27	0	6	1.00	3b	24	0	9	1.00
	LLDPE-S	Cohesive Soil	3c	11	12.4	12	0.94	3d	12	3.7	9	0.93
	LLDPE-S	NW-NP GT	3e	10	0	23	0.63	3f	9	0	23	0.49
	LLDPE-S	Geonet	3g	11	0	9	0.99	3h	10	0	9	1.00
	LLDPE-T	Granular Soil	4a	26	7.7	12	0.95	4b	25	5.2	12	0.95
	LLDPE-T	Cohesive Soil	4c	21	5.8	12	1.00	4d	13	7.0	9	0.98
CAP	LLDPE-T	NW-NP GT	4e	26	8.1	9	1.00	4f	17	9.5	9	0.96
	LLDPE-T	Geonet	4g	15	3.6	6	0.97	4h	11	0	6	0.98
	PVC-S	Granular Soil	5a	26	0.4	6	0.99	5b	19	0	6	0.99
	PVC-S	Cohesive Soil	5c	22	0.9	11	0.88	5d	15	0	9	0.95
	PVC-S	NW-NP GT	5e	20	0	89	0.91	5f	16	0	83	0.74
	PVC-S	NW-HB GT	5g	18	0	3	1.00	5h	12	0.1	3	1.00
	PVC-S	Woven GT	5i	17	0	6	0.54	5j	7	0	6	0.93
	PVC-S	Geonet	5k	18	0.1	3	1.00	5l	16	0.6	3	1.00

Attachment I

NAVFAC DM 7.02 Table 1

AECOM

TABLE 1
Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression: At 1.4 tsf (20 psi)	Typical Value of Compression: At 3.6 tsf (50 psi)	Typical Strength Characteristics: Cohesion (as compacted) psf	Typical Strength Characteristics: Cohesion (saturated) psf	Typical Strength Characteristics: φ (Effective Stress Envelope Degrees)	Typical Strength Characteristics: Tan φ	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
				Percent of Original Height								
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	.....	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-7}$	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 - 150
OH	Organic clays and silty clays	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.
   (..) indicates insufficient data available for an estimate.

Attachment J

Zone of Acceptable Values

AECOM

MINIMUM REQUIRED SHEAR STRENGTH FOR BOTTOM LINER VENEER STABILITY

The following chart depicts the minimum required shear strength values along the interfaces of the engineered components of the bottom liner system under relatively light uniform loads up to the reference load as illustrated in **Figure A.1**. The envelope depicts the peak shear strength used in the veneer stability analyses represented in the form of the minimum friction angle (ϕ) required to maintain the required factors of safety in the veneer stability analysis. All engineered components of the bottom liner system located adjacent to an interface along the slope should have a combination of shear strength parameters (i.e. cohesion/adhesion and friction angle, ϕ) where the mininmum strength for a given normal stress exceeds that of the peak shear strength envelope depicted in **Figure A.1.** Because the normal stress in the veneer analysis is due to a uniform load from the overlying material, a reference stress representing the commensurate normal stress applied from those materials is included on the chart for use in determining appropriate confining pressure for laboratory testing.

Using this reference stress, the acceptable combination of cohesion/adhesion and friction angle are plotted on **Figure A.2.** Any combination plotting above and to the right of the peak value curve represents acceptable values for this application as long as the field loads are equal to or less than the reference stress.

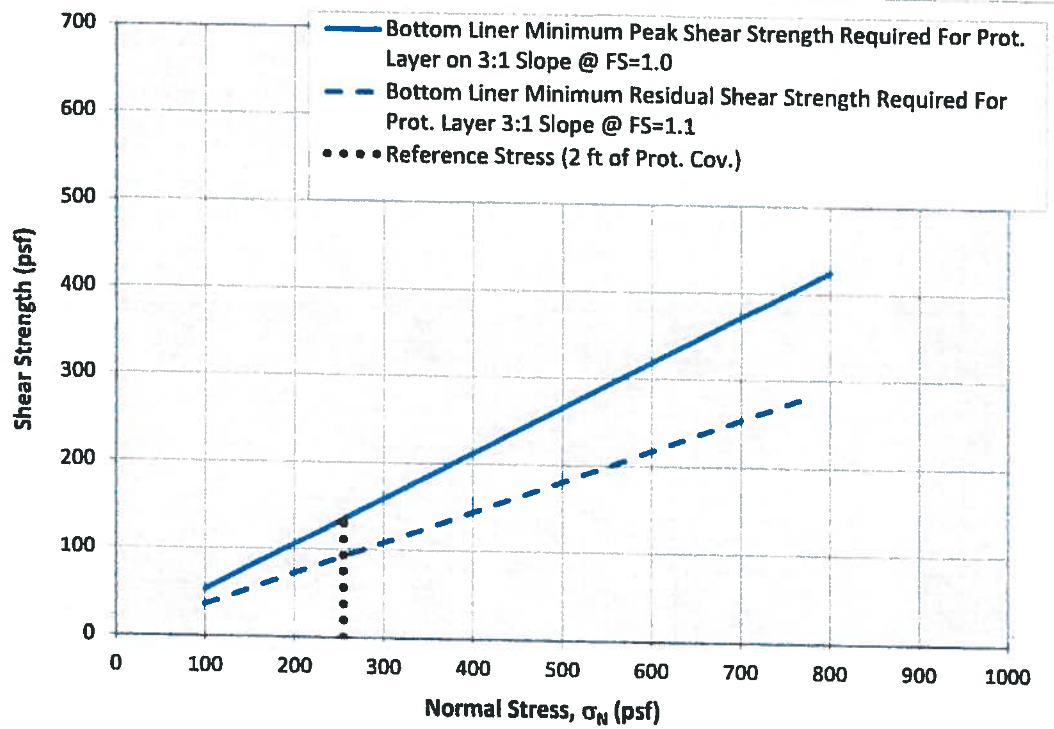

FIGURE A.1

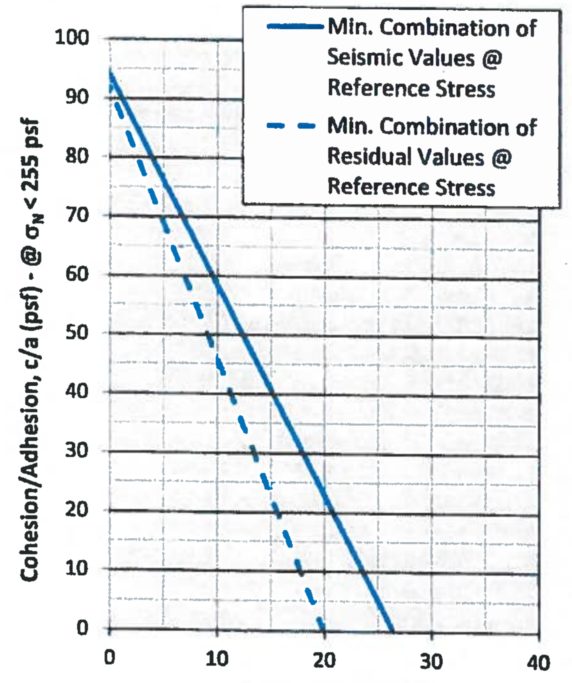

Figure 2: Bottom Liner Zone of Acceptable Values Curves

MINIMUM REQUIRED SHEAR STRENGTH FOR GLOBAL STABILITY

The following chart depicts the minimum required shear strength values along the interfaces of the engineered components of the liner system under relatively light uniform loads up to the reference load as illustrated in **Figure B.1**. The envelope depicts the peak shear strength used in the veneer stability analyses represented in the form of the minimum friction angle (ϕ) required to maintain the required factors of safety in the stability analysis. All engineered components of the final cover system located adjacent to an interface along the slope should have a combination of shear strength parameters (i.e. cohesion/adhesion and friction angle, ϕ) where the mininmum strength for a given normal stress exceeds that of the peak shear strength envelope depicted in **Figure B.1.** Because the normal stress in the veneer analysis is due to a uniform load from the overlying material, a reference stress representing the commensurate normal stress applied from those materials is included on the chart for use in determining appropriate confining pressure for laboratory testing.

Using this reference stress, the acceptable combination of cohesion/adhesion and friction angle are plotted on **Figure B.2.** Any combination plotting above and to the right of the peak value curve represents acceptable values for this application as long as the field loads are equal to or less than the reference stress.

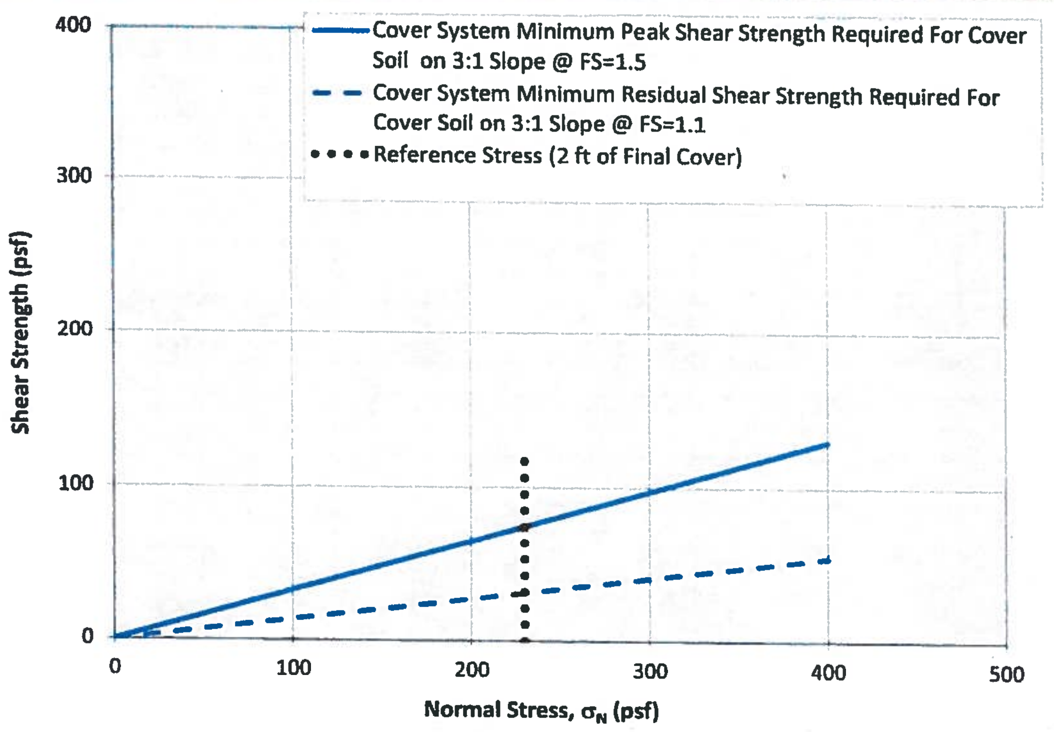

FIGURE B.1

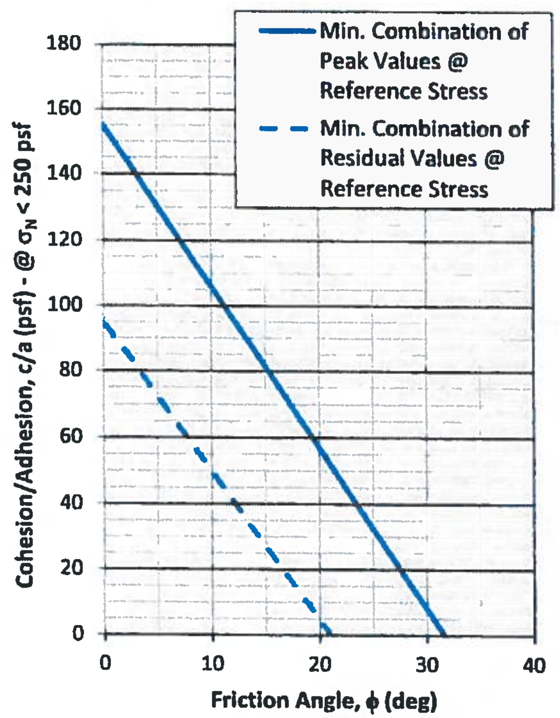

Figure 1: Cover Zone of Acceptable Values Curve

APPENDIX F.4D

Seismic Coefficient Calculation (Rev 3)

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	1 Of 4
Description	Seismic Coefficient Calculation (Rev 3)	Computed by	NCC	Date	1/15/17
		Checked by	MW/NG	Date	9/24/17

I. PURPOSE

The purpose of this analysis is to evaluate the seismic coefficient for use in pseudostatic stability calculations used in the design of the proposed Class II Waste Facility at Scepter in Waverly, Tennessee. The Tennessee Department of Environment and Conservation (TDEC) Solid Waste Processing and Disposal Regulations do not provide prescriptive standards related to liquefaction analysis. These calculations are being provided pursuant to rule 400-11-01-.04(2)(a)3 which states the facility must be located, designed, constructed, and maintained, and closed in such a manner as to minimize to the extent practicable the potential for releases of solid waste, solid waste constituents, or other potentially harmful material to the environment except in a manner authorized by state law.

II. SITE AND PROJECT DESCRIPTION

The seismic coefficient calculations were performed as part of the industrial waste permit application for the Scepter Site located in Waverly, Tennessee.

The proposed site will be permitted as a new industrial solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management.

The following sections summarize the methodology, assumptions, and results of the seismic coefficient calculations for the facility area. For further detail on the specific calculations performed, refer to the corresponding data provided in the **Attachments**.

III. SUMMARY OF SUBSURFACE CONDITIONS

In support of the liquefaction analysis, a hydrogeologic exploration program was performed recently consisting of 7 borings and 2 monitoring wells within and around the proposed footprint of the facility and constituting approximately 494 feet of drilling. In addition, previously developed subsurface data within and in the vicinity of the proposed landfill footprint including over 15 additional borings and laboratory test data were utilized in the assessment. Detailed observations, soil sample test results, analyses, and conclusions concerning the subsurface conditions are presented in the Hydrogeologic Evaluation report for the permit. Additional information was obtained from a subsurface exploration performed by GA Technical Services in 1989. The following summarizes the in-situ conditions related to the settlement analysis.

The subsurface soil material generally consists of residuum consisting of low to medium plasticity clay (CL), clayey silt (ML), clayey sand (SC), and silty sand (SM) based on the Unified Soil Classification System (USCS). Laboratory tested samples obtained from the site typically had between 27 to 73 percent of material by weight passing the #200 sieve with 20% or more finer than 0.005 mm. The plasticity index of the soils generally ranged between 5 and 15 percent indicating a cohesive material of low to medium plasticity. It should be stated that significant quantities of chert gravel were present in the soil matrix of the majority of samples tested. When corrected for gravel content, the effective percent fines of the samples tended to increase by 10% to 40%. As a result, it is expected that the majority of the overburden will behave as a cohesive material regardless of USCS classification. Natural moisture typically varied between 6 and 41 percent. Hydraulic conductivity test results varied from 4.7×10^{-7} cm/s to 7.5×10^{-7} cm/s for samples obtained from relatively undisturbed Shelby tubes.

Generally, weathered bedrock immediately underlies the unconsolidated soil material at the site. Sufficient data for bedrock elevations does not currently exist for the site, therefore, the depth to bedrock across the footprint is assumed to vary from 110 to 130 feet below ground surface (bgs). The uppermost bedrock is comprised primarily of the Fort Payne formation overlying the Chattanooga shale. These upper units are generally described as follows:

- The Fort Payne formation in the area constitutes the upper 100 to 200 feet of the ridges. This formation has weathered almost completely to a residuum of a chert silt/clay matrix and colluvium.

- The Chattanooga Shale is black, fissile, pyritic, carbonaceous often called bituminous shale which acts as a confining unit. The Chattanooga shale does not outcrop at the site and was encountered during drilling for the Scepter facility supply well at a depth of approximately 130 feet below ground surface (bgs).

SEISMIC COEFFICIENT CALCULATION - METHODOLOGY

The seismic coefficient (k_S) is calculated based on the seismic hazard identified at the site. It is a variable in which the inertia forces due to earthquake shaking are represented by a constant horizontal force equal to the weight of the potential sliding mass multiplied by the peak average acceleration of the failure mass. This additional force is used in the limit equilibrium stability analyses to account for seismic impacts in the design for the facility and minimize impacts to the engineered components. This approach is commonly called a pseudostatic analysis and is one of the simplest means used in earthquake engineering to analyze the seismic response of soil embankments and slopes. The seismic coefficient is typically the only variable necessary to be determined to perform the pseudostatic analysis and is used directly in the limit equilibrium analyses in a manner similar to the static analysis. Field observations indicate that the pseudostatic method is appropriate for evaluating the performance of embankments constructed of fill materials that do not lose significant strength during earthquakes such as clays, clayey soils, dry or moist cohesionless soils, and dense cohesionless soils (Seed, 1985). Selection of an appropriate seismic coefficient is the most important, and difficult, aspect of a pseudostatic stability analysis (Kramer, 1997). Because soil slopes are not rigid and the peak acceleration generated during an earthquake last for only a very short period of time, seismic coefficients used in practice generally correspond to acceleration values well below the predicted peak accelerations.

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	3 Of 4
Description	Seismic Coefficient Calculation (Rev 3)	Computed by	NCC	Date	1/15/17
		Checked by	MW/NG	Date	9/24/17

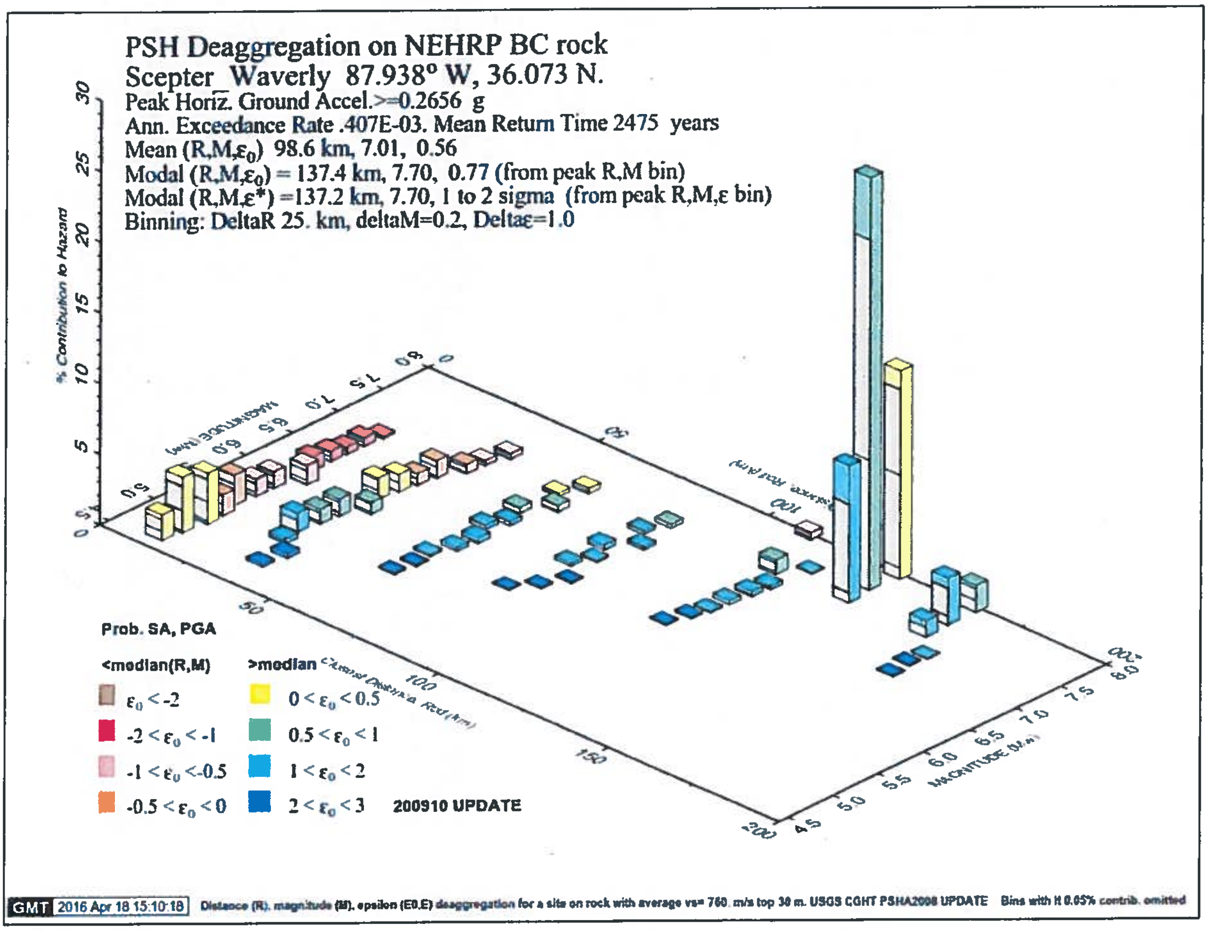

Figure 1. Empirical relationships to characterize ground motions

In the United States, the US Geological Survey (USGS) has developed seismic hazard maps that define the probability of ground motion parameters being exceeded for various levels of uncertainty in the US including ground motion parameters at a 2% probability of exceedance in 50 years (essentially equivalent to 10% probability of exceedance in 250 years) representing a high intensity earthquake and a 10% probability of exceedance in 50 years representing a lower intensity, more frequent earthquake. Figure 1 demonstrates the peak ground acceleration and ground motion parameters at the project site at a 2% probability of exceedance in 50 years, which is a horizontal ground motion of 0.266g. Based on RCRA Subtitle D (258): Seismic Design Guidance for Municipal Solid Waste Facilities (US EPA, 1995) and correspondence with representatives of TDEC, this value is the maximum acceleration (a_{max}) at the crest of the landfill for this project. Further, the acceleration at the base of the landfill for the liner will be 50% of a_{max}.

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	4 Of 4
Description	Seismic Coefficient Calculation (Rev 3)	Computed by	NCC	Date	1/15/17
		Checked by	MW/NG	Date	9/24/17

IV. SEISMIC COEFFICIENT CALCULATION RESULTS

The results are summarized in **Table 1** below.

Table 1. Recommended Seismic Coefficients for Use in Pseudostatic Stability Analysis

Base Liner Seismic Coefficient ($k_{S\text{-}LINER}$)	
$k_{S\text{-}LINER}$	$= a_{max} \times 0.5$
	$= (0.266g)(0.5)$
	$= 0.133g$
Cover Seismic Coefficient ($k_{S\text{-}COVER}$), ($k_{S\text{-}CREST}$)	
$k_{S\text{-}COVER}$	$= a_{max} = 0.266g$
	= (from USGS)

V. CONCLUSIONS

Use of the seismic coefficient (k_S) of 0.133g for base liner stability analysis is acceptable and conservative. The global stability analyses will utilize a seismic coefficient of 0.266g in the seismic stability analyses. This value will also be conservatively assumed as the seismic coefficient for use in the veneer slope analysis for the bottom liner system.

VI. REFERENCES

RCRA Subtitle D (258): Seismic Design Guidance for Municipal Solid Waste Facilities, US EPA, 1995.

VII. ATTACHMENTS

Attachment A	USGS Deaggregation Data for 2% Probability of Exceedance in 50 years
Attachment B	USGS Deaggregation Data for 10% Probability of Exceedance in 50 years

APPENDIX F.4E

Hydrostatic Uplift Analysis (Rev 3)

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	1 Of 4
Description	Hydrostatic Uplift Analysis (Rev 3)	Computed by	SD	Date	06/07/16
		Checked by	MW	Date	07/01/17

I. PURPOSE

The purpose of this analysis is to evaluate the potential for hydrostatic uplift on the liner system at the proposed Class II Disposal Facility at Scepter in Waverly, Tennessee. Uplift calculations are required pursuant to rule 400-11-01-.04(4)(a)1.(iii) which states the liner must be placed upon a foundation or base capable of providing support to the liner and resistance to pressure gradients above and below the liner to prevent failure of the liner due to settlement, compression, or uplift.

Hydrostatic uplift is defined as the tendency of a fluid to exert an upward force on a less permeable surface above the fluid surface. An evaluation will be made to show the facility foundation or base will be capable of providing support to the liner and resistance to pressure gradients above and below the liner to prevent failure of the liner due to uplift.

II. SITE AND PROJECT DESCRIPTION

The hydrostatic uplift analysis was performed as part of the Part II Solid Waste Permit Application for the Scepter Site located in Waverly, Tennessee.

The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management.

The following sections summarize the methodology, assumptions, and results of the hydrostatic uplift analysis.

III. SUMMARY OF SUBSURFACE CONDITIONS

In assessing hydrostatic uplift, a hydrogeologic exploration program was performed recently consisting of 7 borings and 2 monitoring wells within and around the proposed footprint of the facility and constituting approximately 494 feet of drilling. In addition, previously developed subsurface data within and in the vicinity of the proposed landfill footprint including over 15 additional borings and laboratory test data were utilized in the assessment. Detailed observations, soil sample test results, analyses, and conclusions concerning the subsurface conditions are presented in the Hydrogeologic Evaluation report for the permit. Additional information was obtained from a subsurface exploration performed by GA Technical Services in 1989. The following summarizes the in-situ conditions related to the analysis.

a. Soil and Bedrock

The subsurface soil material generally consists of silty clay residuum with some clayey silt, clayey sand, and silty sand. Laboratory tested samples obtained from the site typically had between 27 to 73 percent of material by weight passing the #200 sieve with 20% or more finer than 0.002

mm. The plasticity index of the soils generally ranged between 5 and 15 percent indicating a cohesive material of low to medium plasticity. Natural moisture typically varied between 6 and 41 percent. Hydraulic conductivity test results were typically around 4.7×10^{-7} cm/s to 7.5×10^{-7} cm/s for in-situ Shelby tube samples.

Generally, weathered bedrock immediately underlies the unconsolidated soil material at the site. Depth to bedrock across the footprint varies from approximately 110 feet to 130 feet. The uppermost bedrock is comprised primarily of the Fort Payne formation overlying the Chattanooga shale. These upper units are generally described as follows:

- The Fort Payne formation in the area constitutes the upper 100 to 200 feet of the ridges. This formation has almost completely weathered to a residuum of a chert silt/clay matrix and colluvium.
- The Chattanooga Shale is black, fissile, pyritic, carbonaceous often called bituminous shale which acts as a confining unit. The Chattanooga shale does not outcrop at the site and was encountered during drilling for the Scepter facility supply well at a depth of approximately 130 feet below ground surface (BGS).

b. Groundwater

Groundwater generally occurs in two zones at the site: shallow systems in the alluvial deposits beneath the area at depths ranging from approximately 35 to 80 feet BGS and the deep regional system within the carbonate aquifer in the Fort Payne formation.

The shallow water is semi-confined and its connection to the surface (i.e., recharge by surface infiltration) is evidenced by temperature and water level fluctuations in wells shortly following precipitation events. Local groundwater flow typically mimics the topography, and the flow direction is generally toward area drainage features. A topographically low area is present north of the Scepter landfill.

No ponds, springs, sinkholes, ephemeral seeps or wetlands were identified during site reconnaissance. A perennial water feature exists on the north side of the CSX railroad tracks which is identified as a blue line stream within Leach Hollow on a USGS topographic map. This is the receiving stream for all site drainage which flows west to the Tennessee River.

The regional system in the vicinity of the sites is located within the carbonate aquifer. Groundwater within the regional aquifer is encountered in the Fort Payne formation. Recharge to the groundwater occurs primarily from infiltration of precipitation. The recharge waters ultimately migrates downgradient toward the Tennessee River. The Tennessee River flows south to north adjacent to the Scepter site. It is dammed to form Kentucky Lake near Lake City, Kentucky. Groundwater is expected to be present at depths of 5 to 10 feet below the bottom of the excavcation for the landfill.

IV. HYDROSTATIC UPLIFT ANALYSIS - METHODOLOGY

Facilities should be designed to have a Factor of Safety (FS) against hydrostatic uplift greater than 1.4. The FS is generally determined using the following equation:

$$FS \geq \frac{\sigma_{down}}{\sigma_{up}}$$

Where:

σ_{down} = Downward confining pressure from the overlying material

σ_{up} = Upward hydrostatic pressure

For a section to be stable against uplift, the computed FS must be greater than or equal to 1.4.

V. CALCULATIONS

The factor of safety against hydrostatic uplift is calculated as follows:

$$FS = \frac{\gamma_S(t_S) + \gamma_{GB}(t_{GB})}{\gamma_W \Delta h}$$

Where:

t_S	=	Thickness of Overlying In Situ Overlying Confining Layer
γ_S	=	Wet Unit Weight of Overlying Confining Soil, 120 pcf (assumed)
t_{GB}	=	Thickness of Geologic Barrier Layer, 10 ft (min required)
γ_{GB}	=	Wet Unit Weight of Geologic Barrier , 120 pcf (assumed)
γ_W	=	Unit Weight of Water, 62.4 pcf
Δh	=	Piezometric Head Associated with Hydrostatic Uplift Pressure
FS	=	Minimum Recommended Factor of Safety, 1.4

For the case where the groundwater is below the excavation limits, the piezometric head associated with hydrostatic uplift pressure is zero. As a result, the factor of safety against hydrostatic uplift cannot be evaluated. As the piezometric head approaches zero, the factor of safety against hydrostatic uplift approaches infinity.

VI. ANALYSIS AND CONCLUSIONS

As discussed above, the groundwater surface elevation at the has been determined through potentiometric modeling calibated to field groundwater measurements associated with multiple monitoring wells installed across the property. Groundwater at the project site is expected to be approximately 5 to 10 feet below the bottom of the excavation. Accordingly, the piezometric head will be or approach zero, resulting in a factor of safety against hydrostatic uplift appoaching infitinity. In addition, there is no true confining layer, as the foundation soils, geoglogic buffer, and compacted soil liner each consist of the same material. Bedrock is expected to be well below the bottom of the excavation. As such, hydrostatic uplift is not expected to develop, and the proposed liner is expected to be stable in this regard.

APPENDIX F.4F

Settlement Analysis (Rev 3)

APPENDIX F.4F

Settlement Analysis

AECOM

I. PURPOSE

The purpose of this analysis is to evaluate the settlement potential of the foundation soils for the proposed Scepter Inc. Waverly Class II Disposal Facility. Settlement calculations are required pursuant to rule 400-11-01-.04(4)(a)1.(iii) which states the liner must be placed upon a foundation or base capable of providing support to the liner and resistance to pressure gradients above and below the liner to prevent failure of the liner due to settlement, compression, or uplift.

II. SITE AND PROJECT DESCRIPTION

The settlement analysis was performed as part of the Part II Solid Waste Permit Application for the landfill facility for Scepter, Inc. in Waverly, Tennessee.

The proposed site will be permitted as a new Class II solid waste landfill through the Tennessee Department of Environment and Conservation (TDEC) Division of Solid Waste Management.

The following sections summarize the methodology, assumptions, and results of the settlement analysis. For further detail on the specific calculations performed, refer to the corresponding data provided in the Attachments.

III. SUMMARY OF SUBSURFACE CONDITIONS

In assessing hydrostatic uplift, a hydrogeologic exploration program was performed recently consisting of 7 borings and 2 monitoring wells within and around the proposed footprint of the facility and constituting approximately 494 feet of drilling. In addition, previously developed subsurface data within and in the vicinity of the proposed landfill footprint including over 15 additional borings and laboratory test data were utilized in the assessment. Detailed observations, soil sample test results, analyses, and conclusions concerning the subsurface conditions are presented in the Hydrogeologic Evaluation report for the permit. Additional information was obtained from a subsurface exploration performed by GA Technical Services in 1989. The following summarizes the in-situ conditions related to the settlement analysis.

a. Soil and Bedrock

The subsurface soil material generally consists of silty clay residuum with some clayey silt, clayey sand, and silty sand. Laboratory tested samples obtained from the site typically had between 27 to 73 percent of material by weight passing the #200 sieve with 20% or more finer than 0.002 mm. The plasticity index of the soils generally ranged between 5 and 15 percent indicating a cohesive material of low to medium plasticity. Natural moisture typically varied between 6 and 41 percent. Hydraulic conductivity test results were typically around 4.7×10-7 cm/s to 7.5×10-7 cm/s for in-situ Shelby tube samples.

Generally, weathered bedrock immediately underlies the unconsolidated soil material at the site. Depth to bedrock across the footprint varies from approximately 110 feet to 130 feet. The uppermost bedrock is comprised primarily of the Fort Payne formation overlying the Chattanooga shale. These upper units are generally described as follows:

- The Fort Payne formation in the area constitutes the upper 100 to 200 feet of the ridges. This formation has almost completely weathered to a residuum of a chert silt/clay matrix and colluvium.
- The Chattanooga Shale is black, fissile, pyritic, carbonaceous often called bituminous shale which acts as a confining unit. The Chattanooga shale does not outcrop at the site and was encountered during drilling for the Scepter facility supply well at a depth of approximately 130 feet below ground surface (BGS).

b. Groundwater

Groundwater generally occurs in two zones at the site: shallow systems in the alluvial deposits beneath the area at depths ranging from approximately 35 to 80 feet BGS and the deep regional system within the carbonate aquifer in the Fort Payne formation.

The shallow water is semi-confined and its connection to the surface (i.e., recharge by surface infiltration) is evidenced by temperature and water level fluctuations in wells shortly following precipitation events. Local groundwater flow typically mimics the topography, and the flow direction is generally toward area drainage features. A topographically low area is present north of the Scepter landfill.

No ponds, springs, sinkholes, ephemeral seeps or wetlands were identified during site reconnaissance. A perennial water feature exists on the north side of the CSX railroad tracks which is identified as a blue line stream within Leach Hollow on a USGS topographic map. This is the receiving stream for all site drainage which flows west to the Tennessee River.

The regional system in the vicinity of the sites is located within the carbonate aquifer. Groundwater within the regional aquifer is encountered in the Fort Payne formation. Recharge to the groundwater occurs primarily from infiltration of precipitation. The recharge waters ultimately migrates downgradient toward the Tennessee River. The Tennessee River flows south to north adjacent to the Scepter site. It is dammed to form Kentucky Lake near Lake City, Kentucky.

IV. COMPONENTS OF SETTLEMENT

Under load, soils settle. This can cause settlement of structures located on or within the soil. Some degree of settlement is inevitable, but the settlement must be kept to a tolerable limit or the desired use or design life of the structure may be impaired.

Structures may settle uniformly or nonuniformly. Nonuniform settlement is called differential settlement and is the primary concern of the settlement analysis. Once total settlement is computed, differential settlement can also be computed by comparing total settlement at two adjacent locations and calculating the difference.

Total settlement usually consists of three parts—immediate or elastic compression, primary consolidation, and secondary compression.

a. Immediate Settlement

Immediate settlement generally takes place as the load is applied or within a time period of about 7 days and predominates in cohesionless soils and unsaturated clay. Immediate settlement usually isn't calculated for fine grain materials that have a degree of saturation of approximately 90% or more.

Based on proposed geologic buffer elevation, the groundwater surface is expected to be within 5 feet of the top of geologic buffer. So, foundation soils are expected to be saturated once the subgrade elevation is reached. Therefore, immediate settlement is neglected for this analysis.

b. Primary Consolidation

Of the three contributors, primary consolidation accounts for the majority of settlement. For fine grained soils, primary consolidation involves the expulsion of water from the pore spaces resulting in a volume reduction as the soil particles rearrange themselves to form a denser configuration. This process is controlled by the hydraulic conductivity of the material and can therefore take a long period of time for fine grained materials.

Because consolidation settlement is responsible for nearly all the total settlement, it is the focus of this analysis and is explained in more detail below.

c. Secondary Compression

Secondary compression occurs through volume changes as in primary consolidation except that it occurs at a much slower rate and occurs via creep through crushing of inter-granular particles. This type of settlement occurs at a constant effective stress (i.e. once the excess pore water is dispersed). Secondary compression is unlikely to contribute significantly to the overall total settlement for inorganic, overconsolidated soils. Because primary settlement can be estimated only within 10 to 20 percent accuracy and because the foundation and engineered components are somewhat tolerant of differential settlement, secondary settlement is neglected for this analysis.

It should be noted that the above procedure assumes all foundation soils are cohesive and will be subject to primary and secondary consolidation only. However, in reality, intervals of the foundation soils are granular and will likely undergo elastic settlement as waste is placed. As such, post construction settlement will likely be less than the model predicts. Accordingly, the settlement results are expected to be conservative.

V. MODEL THEORY

Every natural soil is characterized by its composition and structure. Many combinations of composition and structure allow soil to exist at different void ratios allowing for both small and large responses to external loads. Most settlement problems do not alter the composition of a soil, and compression takes place exclusively in response to an increase in effective vertical stress due to load. The natural soil structure adjusts to a new orientation undergoing a decrease in the volume of voids. The compression response to the initial increments of effective stress is strongly influenced by the natural structure of the soil.

Consolidation theory is often illustrated by use of a spring analogy where the characteristic of the spring represent the characteristics of the soil structure. As the spring is compressed (i.e. the soil skeleton), water is forced out of the soil pores. The spring in this case has a "memory" where after the load is removed, the spring rebounds, but not to the initial starting point. Instead it rebounds more slowly and returns to some interim point. If the spring were to be compressed again beyond the initial maximum stress, it is less compressible at first because of the stress history. However, as the pore water flows away from the soil and it once again reaches the conditions equivalent to the maximum past pressure, it becomes more compressible returning to its initial characteristics.

This circumstance where a soil has been historically compressed under a higher load than current conditions is called overconsolidation. Geologically, the best-known process for causing overconsolidation is preloading, in which the increased resistance to compression is the result of previous consolidation of the clay under past effective vertical pressures higher than that of the existing effective overburden. The historical maximum stress is called the preconsolidation pressure (σ'_P or p_C) and it defines the boundary between stiff and soft deformation response of a soil to loading. Soils where the in-situ stress is equivalent to the maximum load are called normally consolidated.

Preconsolidation can be measured via laboratory oedometer testing. The same test can measure the ratio of deformation to load that occurs under stress less than the preconsolidation stress which is called the recompression index (c_R). Similarly, this test can measure the ratio of deformation to load that occurs relative to the preconsolidation stress along a compression curve as depicted in Figure 1. This figure depicts theoretical test results for both normally consolidated and overconsolidated clay samples. The recompression index (c_R) is equivalent to the slope of the compression curve prior to the preconsolidation pressure and the compression index (c_C) is equivalent to the slope of the compression curve after the preconsolidation stress. The field compression curves depicted are generated based on the procedures suggested by Schmertmann (1955).

The magnitude of the preconsolidation pressure is best expressed in terms of the ratio of in-situ effective stress (σ_{VO}) to the preconsolidation stress value of (σ_{VO}/σ'_P) known as the overconsolidation ratio, OCR.

Measured magnitudes of OCR for a variety of natural soft clays are in the range of 1.2 to 3 except within the desiccated crust, where they may be much higher.

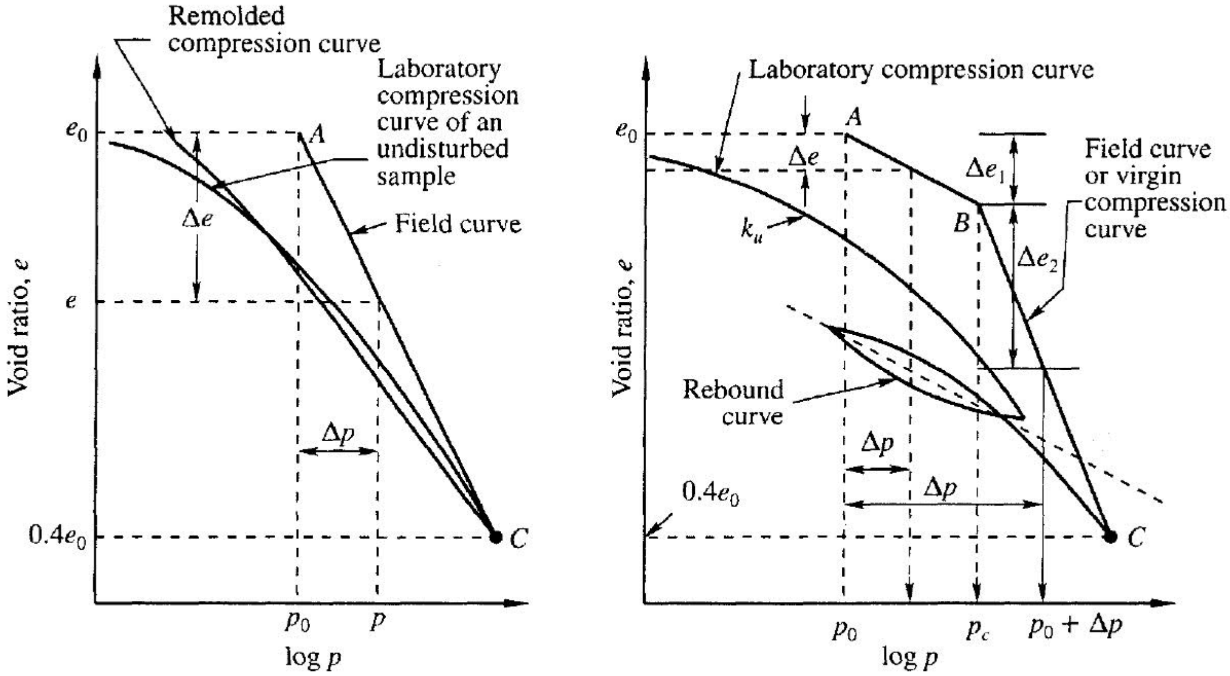

Figure 1. Normally consolidated and overconsolidated test results and corresponding estimated field curves

By use of the unique compression curve and the settlement parameters associated with a given soil, the change in void ratio (and the associated settlement) can be determined in response to a surcharge load ($\Delta\sigma_V$) using the following governing equations.

For the normally consolidated case, where the preconsolidation pressure is equal to the in-situ pressure, the following equation applies.

$$s_C = \frac{H_0 C_c}{1+e_0} log \frac{\sigma'_{V0} + \Delta\sigma_V}{\sigma'_{V0}}$$

Where:

s_c = Settlement of layer

H_0 = Thickness of layer

C_c = Compression Index of layer

e_0 = Initial void ratio at layer

σ'_{VO} = Effective overburden pressure at layer center

$\Delta\sigma_V$ = Surcharge pressure at layer center

There are two possible cases related to degree of overconsolidation. For the first possible overconsolidated case, where the sum of the vertical overburden pressure and the surcharge pressure of the layer is greater than the preconsolidation pressure, the following equation applies.

$$s_C = \frac{H_0 C_c}{1+e_0} log \frac{\sigma'_{V0} + \Delta\sigma_V}{\sigma'_P} + \frac{H_0 C_R}{1+e_0} log \frac{\sigma'_P}{\sigma'_{V0}}$$

Where:

σ_P' = Effective preconsolidation pressure

C_R = Recompression Index of layer

For the second possible overconsolidated case where the sum of the vertical overburden pressure and the surcharge pressure of the layer is less than the preconsolidation pressure, the following equation applies:

$$s_C = \frac{H_0 C_R}{1+e_0} log \frac{\sigma'_{V0} + \Delta\sigma_V}{\sigma'_{V0}}$$

The surcharge pressure applied to the settlement calculations was determined using a generalized embankment load for each cross section. The surcharge pressure due to embankment loading was determined using the following calculations from Das (2010). For this two-dimensional loading condition, the surcharge pressure is calculated using the following equation and Figure 2.

$$\Delta\sigma_z = \frac{q_o}{\pi}\left[\left(\frac{B_1 + B_2}{B_2}\right)(\alpha_1 + \alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]$$

Where:

$q_o = \gamma H$

g = Unit weight of embankment soil

H = Height of embankment

$$\alpha_1(radians) = \tan^{-1}\left(\frac{B_1 + B_2}{B_2}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right)$$

z = depth to midpoint of foundation soil layer

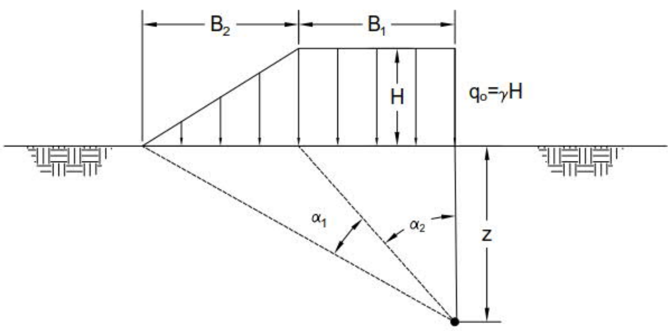

Figure 2. Embankment loading

The equation for embankment loading was adjusted according to each cross section and point of interest. For embankments with a grade break, the equations were adjusted to calculate two embankment loads and the total surcharge load was obtained by the superposition principle. The calculations for the embankment loading are provided in Attachment D.

VI. SETTLEMENT METHODOLOGY

Classical settlement theory was used in all settlement analysis performed. Overall settlement, differential settlement, and resulting strains were analyzed. The following sections summarize the settlement considerations, procedures, interpretation of subsurface conditions, assumptions, results and conclusions of the settlement analysis. For further detail on the specific calculations performed, refer to the corresponding data provided in the Attachments.

a. Settlement Considerations

Based on laboratory testing results and geologic history, the majority of the material (residual clayey soil) is slightly overconsolidated, meaning that the total in-situ pressure is less than the preconsolidation pressure. As discussed, it is assumed settlement will occur primarily as a result of consolidation of the in-situ clay.

The proposed waste material will likely be moderately compacted during placement. Significant settlement of the waste material is not anticipated. Furthermore, any settlements in the waste itself will not affect the engineered components of the landfill liner system. Thus, settlement of the waste is not considered in this analysis.

b. Settlement Methodology

The overall steps/elements of the settlement analysis are as follows:

- Locate cross-sections along which settlements are to be computed and develop subsurface profiles at defined intervals represented by points;
- Choose material parameters for the points along each settlement section, including the surcharge imposed by the fill, as described in Material Parameters; and
- Compute the settlement for each point on the Section line using the applicable settlement equations described above.

VII. CROSS SECTION LOCATION

Cross sections were determined strategically to evaluate the effects of total and differential settlement on the engineered components of the liner and leachate collection system. However, it should be noted that cross sections do not exactly align with proposed leachate collection pipes, nor are the cross sections aligned exactly perpendicular to the liner. The pre-settlement slope of the liner floor to the sump is 3.6%, and the pre settlement slope of the proposed leachate lines is approximately 2.7% to 3%.

a. Stratigraphy

Based on results of the subsurface exploration program and design of the facility, a stratigraphic model (from the top down) was developed for the settlement analysis as shown in Table 1.

Table 1. Settlement Stratigraphy

Layer	Soil Layer	Thickness	Description
1	Final Cover System	3 ft	Layer includes vegetative and protective soil components, Geocomposite drainage, FML barrier, and geocomposite gas pressure relief. Layers are modeled as a single composite with a conservative unit weight.
2	Waste Material	Varies	Layer includes aluminum salt cake waste material only. Thickness varies throughout facility.
3	Bottom Liner System	2 ft	Layer includes protective layer, geosynthetic drainage, and geomembrane liner. Layers are modeled as a single composite with a conservative unit weight.
4	Compacted Clay Liner	2 ft	Layer is composed of recompacted soil from on-site materials to be constructed above geologic buffer/native soils.
5	Geologic Buffer/Native Clay Soil	Varies	Layer is composed of native residuum/alluvial soil. Material identified in subsurface exploration as clay to silty sand describes the majority of onsite soil. Conservative settlement parameters were assigned based on consolidation testing from onsite samples for silty clay and clayey silt.
6	Bedrock	Infinite	Depth and location of bedrock is based on previous subsurface explorations and varies across the site. Top of bedrock in the model represents areas where refusal was reached during split spoon sampling in previous subsurface explorations. Bedrock is considered incompressible for purposes of settlement and therefore strength and thickness are modeled as infinite.

b. Water Table

The piezometric surface representing the perched water is based on a conservative representation of perched water levels encountered during subsurface exploration at the site. Perched water levels were encountered at depths of 35 to 80 feet BGS. The water level generally dips towards the northwestern side of the facility from elevation 493 feet to about 430 feet. Due to the underdrain in the design, perched water will remain below the compacted clay liner in all areas.

c. Settlement Sections and Subsurface Profiles

Settlement sections were chosen across the landfill in the south-north and west-east directions. The sections were chosen to reflect the variability in thickness of overburden soils and bedrock topography and to provide appropriate geospatial coverage. The sections were also chosen to encompass the full height of the landfill. The landfill configurations used in the settlement analyses correspond to that given in the Permit Drawings.

Attachment A presents the location of the sections with respect to top of leachate collection layer grades. Equidistant points, approximately spaced no more than 50 feet apart, are depicted along each section numbered in accordance with stationing. Attachment B and Attachment C show the profile along each cross section.

The settlement analysis is based on the material types and properties presented in the Hydrogeologic Evaluation report that includes the results of the geotechnical exploration, and the general summary of subsurface conditions presented above. The pertinent material properties and a summary of the results are presented below.

VIII. MATERIAL PROPERTIES

a. Surcharge Magnitudes

Surcharge load drives consolidation settlement. It is computed by evaluating the embankment loading at the midpoint of the native clay layer for each equidistant point along the profile.

Final Cover System: This layer is modeled as a composite of materials. This layer includes 6 inches of vegetative cover, 18 inches of protective cover soil, a geocomposite drainage layer, a flexible geomembrane layer, a gas pressure relief geocomposite layer, and a foot of barrier soils. The geosynthetic layers are thin such that their weight is negligible. Only the soil layers contribute substantially to the unit weight of the cover system. Total unit weight for the final cover layers is anticipated to range between 100 and 110 pcf.

Waste Materials: The primary contributor to the surcharge load responsible for settlement is the waste product that is to be disposed in the landfill. The waste material includes aluminum salt cake material only. Total unit weight for the waste material is anticipated to range between 100 and 110 pcf.

Liner and Leachate Collection System: This layer is modeled as a composite of materials. This layer includes a foot of barrier soils, a geocomposite drainage layer, a geosynthetic clay liner (GCL), leachate collection pipes, a flexible membrane liner (FML), a protective cover layer of sand and clay, and a geologic buffer of native clay. The geosynthetic layers are thin such that their weight is negligible. The permeability of the native soil allows it to serve as the geologic buffer. Therefore, only the cover layers contribute substantially to the unit weight of this liner system. Total unit weight for the cover layers range between 100 and 120 pcf depending on the confining stress.

The total unit weight of the surcharge materials and thickness anticipated is shown in Table 2 below.

Table 2. Settlement Analysis Input Parameters – Surcharge Materials

Layer	Total Unit Wt., γ_T (pcf)	Thickness (ft)
Final Cover Layer	110	3
Waste Material	110	Varies
Liner/Drainage/Protective/CSL	120	4

b. Primary Settlement Parameters

Material properties pertinent to consolidation settlement analysis include the soil total unit weight, initial void ratio (e_0), preconsolidation pressure (σ'_P), and compression indices (C_C and C_R). The soil properties used in the settlement analysis for the in-situ foundation materials were estimated based on published correlations and consolidation tests. Parameters were assigned to the native clay based on these results. Unit weight (γ_T) and the settlement parameters were held constant in all analyses.

Table 3. Settlement Parameters for Compressible Material

Layer	γ (pcf)	s'$_p$ (psf)	e_0	C_c	C_R
Foundation Soil	120	2600	0.90	0.20	0.04

IX. RESULTS

Due to the large number of computations involved, a spreadsheet program was used to perform the analysis. Spreadsheet outputs for all points analyzed, the differential settlement between points, and the minimum grades calculated along the profile are provided in Attachment D.

Differential settlement and liner strain resulting from the settlement were also computed. Liner strain is calculated between points along the sections. Percent liner strain was calculated as follows:

$$E_T(\%) = \frac{L_f - L_0}{L_0} * 100$$

Where:

E_T = Tensile strain
L_0 = Original distance separating two location points
L_f = Final distance separating the same two points after settlement is complete

The tables below summarize the consolidation settlement, differential settlement, strain, and post settlement grades at each station for each cross section. Calculations for each point are included in Attachment D. A sample calculation of settlement is provided in Attachment E.

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	11 of 18
Description	Settlement Analysis (Rev 3)	Computed by	SD	Date	04/15/18
		Checked by	NC/MW	Date	04/15/18

Table 4. Summary of Settlement Calculations Along South-North Cross Section 1

Station	Max Settlement (in)	Max. Differential Settlement (in)	Max. Strain (%)	Overall Post Settlement Slope (%)
5+33.82	27.26	---	---	---
5+83.82	29.09	1.83	0.0936%	-34.35%
6+33.82	32.56	3.47	0.1781%	-34.76%
6+83.82	42.85	10.30	0.5379%	-35.98%
7+33.82	44.47	1.61	0.0826%	-34.51%
7+83.82	31.08	13.39	0.6640%	-32.03%
8+33.82	29.49	1.59	0.0241%	-9.01%
8+83.82	32.77	3.28	0.0185%	-3.67%
9+33.82	31.59	1.18	0.0059%	-2.90%
9+83.82	32.95	1.37	0.0074%	-3.35%
10+33.82	34.47	1.51	0.0082%	-3.39%
10+83.82	37.52	3.05	0.0173%	-3.65%
11+33.82	42.69	5.16	0.0307%	-4.00%
11+83.82	48.41	5.72	0.0344%	-4.09%
12+33.82	56.43	8.03	0.0509%	-4.48%
12+83.82	52.03	4.40	0.0203%	-2.41%
13+33.82	45.94	6.10	0.0241%	-1.86%
13+83.82	37.41	8.53	0.0277%	-1.24%
14+33.82	31.96	5.45	0.0200%	-1.75%
14+83.82	26.91	5.05	0.0190%	-1.84%
15+33.82	24.19	2.72	0.0767%	17.65%
15+83.82	23.37	0.82	0.0381%	30.52%
16+33.82	24.70	1.33	0.0576%	28.02%

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	12 of 18
Description	Settlement Analysis (Rev 3)	Computed by	SD	Date	04/15/18
		Checked by	NC/MW	Date	04/15/18

Table 5. Summary of Settlement Calculations Along South-North Cross Section 2

Station	Max Settlement (in)	Max. Differential Settlement (in)	Max. Strain (%)	Overall Post Settlement Slope (%)
4+46.42	30.57	---	---	---
4+96.42	31.78	1.21	0.0625%	-34.80%
5+46.42	33.37	1.59	0.0818%	-34.65%
5+96.42	36.10	2.73	0.1394%	-34.41%
6+46.42	41.52	5.42	0.0929%	-10.84%
6+96.42	49.13	7.61	0.0405%	-3.83%
7+46.42	54.51	5.37	0.0271%	-3.48%
7+96.42	46.58	7.92	0.0251%	-1.24%
8+46.42	39.92	6.66	0.0222%	-1.45%
8+96.42	34.04	5.88	0.0205%	-1.60%
9+46.42	30.31	3.73	0.0136%	-1.88%
9+96.42	27.73	2.58	0.0093%	-1.95%
10+46.42	27.44	0.29	0.0011%	-2.25%
10+96.42	30.56	3.11	0.0136%	-2.88%
11+46.42	39.36	8.80	0.0433%	-3.69%
11+96.42	42.01	2.65	0.0110%	-2.72%
12+46.42	36.18	5.83	0.0164%	-1.21%
12+96.42	31.50	4.69	0.0138%	-1.38%
13+46.42	26.91	4.59	0.0136%	-1.39%
13+96.42	23.65	3.25	0.0106%	-1.68%
14+46.42	21.49	2.16	0.0411%	11.74%
14+96.42	19.73	1.77	0.0905%	34.49%

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	13 of 18
Description	Settlement Analysis (Rev 3)	Computed by	SD	Date	04/15/18
		Checked by	NC/MW	Date	04/15/18

Table 6. Summary of Settlement Calculations Along South-North Cross Section 3

Station	Max Settlement (in)	Max. Differential Settlement (in)	Max. Strain (%)	Overall Post Settlement Slope (%)
4+50.00	30.67	---	---	---
5+00.00	32.58	1.91	0.0948%	-33.06%
5+50.00	35.34	2.76	0.1368%	-33.18%
6+00.00	37.88	2.53	0.1256%	-33.14%
6+50.00	41.35	3.48	0.1727%	-33.32%
7+00.00	51.37	10.02	0.1556%	-10.23%
7+50.00	57.97	6.60	0.0342%	-3.66%
8+00.00	51.15	6.82	0.0226%	-1.42%
8+50.00	43.21	7.94	0.0254%	-1.26%
9+00.00	35.89	7.33	0.0238%	-1.34%
9+50.00	34.43	1.46	0.0060%	-2.34%
10+00.00	38.99	4.56	0.0223%	-3.32%
10+50.00	40.01	1.02	0.0045%	-2.75%
11+00.00	42.64	2.64	0.0123%	-3.02%
11+50.00	50.32	7.68	0.0386%	-3.66%
12+00.00	48.34	1.98	0.0070%	-1.95%
12+50.00	44.15	4.19	0.0126%	-1.46%
13+00.00	40.79	3.36	0.0110%	-1.68%
13+50.00	32.47	8.32	0.0198%	-0.73%
14+00.00	26.17	6.30	0.0169%	-1.09%
14+50.00	23.48	2.70	0.1097%	26.29%
15+00.00	22.79	0.69	0.0348%	33.66%

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	14 of 18
Description	Settlement Analysis (Rev 3)	Computed by	SD	Date	04/15/18
		Checked by	NC/MW	Date	04/15/18

Table 7. Summary of Settlement Calculations Along South-North Cross Section 4

Station	Max Settlement (in)	Max. Differential Settlement (in)	Max. Strain (%)	Overall Post Settlement Slope (%)
4+63.00	28.94	---	---	---
5+13.00	31.06	2.11	0.1044%	-33.03%
5+63.00	33.63	2.57	0.1276%	-33.17%
6+13.00	37.09	3.46	0.1716%	-33.30%
6+63.00	49.37	12.28	0.6223%	-34.79%
7+13.00	35.75	13.62	0.6300%	-29.29%
7+63.00	32.31	3.45	0.0130%	-1.99%
8+13.00	33.21	0.90	0.0039%	-2.71%
8+63.00	28.43	4.78	0.0170%	-1.74%
9+13.00	28.49	0.06	0.0003%	-2.57%
9+63.00	30.91	2.42	0.0111%	-2.96%
10+13.00	36.63	5.72	0.0291%	-3.53%
10+63.00	42.65	6.02	0.0309%	-3.58%
11+13.00	51.22	8.57	0.0470%	-4.01%
11+63.00	52.59	1.37	0.0062%	-2.81%
12+13.00	43.15	9.44	0.0282%	-1.01%
12+63.00	36.73	6.42	0.0219%	-1.51%
13+13.00	37.96	1.23	0.0054%	-2.75%
13+63.00	30.43	7.54	0.0192%	-0.90%
14+13.00	24.26	6.17	0.1388%	14.27%
14+63.00	24.19	0.07	0.0034%	33.45%
15+13.00	25.09	0.90	0.0450%	33.31%

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	15 of 18
Description	Settlement Analysis (Rev 3)	Computed by	SD	Date	04/15/18
		Checked by	NC/MW	Date	04/15/18

Table 8. Summary of Settlement Calculations Along West-East Cross Section 1

Station	Max Settlement (in)	Max. Differential Settlement (in)	Max. Strain (%)	Overall Post Settlement Slope (%)
3+02.63	17.71	---	---	---
3+52.63	20.74	3.03	0.1291%	-27.74%
4+02.63	24.33	3.59	0.0130%	1.88%
4+52.63	27.70	3.37	0.0114%	1.76%
5+02.63	30.65	2.95	0.0104%	1.87%
5+52.63	32.31	1.66	0.0062%	2.08%
6+02.63	34.27	1.96	0.0069%	1.95%
6+52.63	35.60	1.33	0.0049%	2.12%
7+02.63	36.93	1.32	0.0052%	2.24%
7+52.63	38.62	1.69	0.0065%	2.18%
8+02.63	39.82	1.20	0.0047%	2.26%
8+52.63	39.47	0.36	0.0015%	2.52%
9+02.63	37.97	1.49	0.0084%	3.51%
9+52.63	35.60	2.37	0.0074%	2.06%
10+02.63	34.74	0.86	0.0037%	2.62%
10+52.63	38.25	3.52	0.0127%	1.87%
11+02.63	39.76	1.51	0.0058%	2.19%
11+52.63	43.63	3.87	0.0137%	1.79%
12+02.63	38.17	5.46	0.0263%	3.35%
12+52.63	30.22	7.95	0.2475%	20.02%
13+02.63	32.13	1.91	0.0973%	33.96%
13+52.63	35.26	3.12	0.1586%	33.74%
14+02.63	38.87	3.61	0.1830%	33.68%
14+52.63	43.30	4.43	0.2245%	33.54%

AECOM

Job	Scepter Waverly Landfill-East Phase	Project No.	60398526	Sheet	16 of 18
Description	Settlement Analysis (Rev 3)	Computed by	SD	Date	04/15/18
		Checked by	NC/MW	Date	04/15/18

Table 9. Summary of Settlement Calculations Along West-East Cross Section 2

Station	Max Settlement (in)	Max. Differential Settlement (in)	Max. Strain (%)	Overall Post Settlement Slope (%)
3+34.00	29.16	---	---	---
3+84.00	31.33	2.16	0.1017%	-31.06%
4+34.00	34.30	2.97	0.0488%	-10.20%
4+84.00	36.92	2.62	0.0094%	1.92%
5+34.00	39.08	2.16	0.0082%	2.10%
5+84.00	41.29	2.20	0.0084%	2.09%
6+34.00	43.27	1.98	0.0076%	2.13%
6+84.00	44.87	1.60	0.0062%	2.17%
7+34.00	45.32	0.45	0.0018%	2.39%
7+84.00	48.37	3.06	0.0112%	1.95%
8+34.00	46.21	2.17	0.0096%	2.84%
8+84.00	41.36	4.84	0.0231%	3.27%
9+34.00	38.13	3.24	0.0146%	2.98%
9+84.00	36.63	1.49	0.0064%	2.71%
10+34.00	28.32	8.31	0.0433%	3.82%
10+84.00	23.04	5.28	0.0253%	3.32%
11+34.00	20.09	2.95	0.0132%	2.93%
11+84.00	17.77	2.32	0.0102%	2.83%
12+34.00	16.21	1.56	0.0067%	2.70%
12+84.00	21.52	5.31	0.2423%	29.44%
13+34.00	27.52	6.00	0.3028%	33.28%
13+84.00	33.00	5.48	0.2766%	33.37%
14+34.00	37.99	4.99	0.2524%	33.43%
14+84.00	41.67	3.68	0.1865%	33.67%

Table 10. Summary of Settlement Calculations Along West-East Cross Section 3

Station	Max Settlement (in)	Max. Differential Settlement (in)	Max. Strain (%)	Overall Post Settlement Slope (%)
6+57.00	22.76	---	---	---
7+07.00	30.56	7.80	0.1650%	-13.54%
7+57.00	36.69	6.13	0.0181%	1.26%
8+07.00	38.73	2.03	0.0071%	1.92%
8+57.00	42.07	3.34	0.0113%	1.74%
9+07.00	45.00	2.93	0.0096%	1.73%
9+57.00	47.40	2.40	0.0079%	1.78%
10+07.00	48.49	1.09	0.0038%	2.02%
10+57.00	49.07	0.58	0.0021%	2.08%
11+07.00	50.72	1.64	0.0056%	1.93%
11+57.00	46.65	4.06	0.0191%	3.16%
12+07.00	43.21	3.44	0.0157%	3.03%
12+57.00	39.18	4.03	0.0189%	3.15%
13+07.00	35.99	3.19	0.0145%	2.99%
13+57.00	28.91	7.09	0.0362%	3.66%
14+07.00	26.49	2.42	0.0423%	10.80%
14+57.00	29.96	3.47	0.1740%	33.16%
15+07.00	33.75	3.79	0.1919%	33.63%
15+57.00	36.53	2.78	0.1413%	33.82%

As indicated by the results, the magnitude of computed settlements is variable with the highest values calculated near the center of the landfill and in areas with thicker layers of native soils. At the cross sections analyzed, which are oriented along a flatter pre settlement slope than the leachate collection pipes, the pre settlement slope varied from 2.1% to 2.6%. At two 50-ft sections along a total of approximately 10,000 linear feet of cross section analyzed, the post construction settlement was less than 1% (0.73% and 0.90%, at Section SN-3, Station 13+50, and at Section SN-4, Station 13+63). However, the pre-settlement slope of the actual leachate collection pipe at these areas varies from 2.7% to 3%. Accordingly, the post settlement slope of the leachate collection pipe in the vicinity of these locations is by observation at least 1%. All other locations analyzed in the vicinity of the proposed leachate collection pipes demonstrated a post settlement slope of at least 1%.

In addition, the greatest magnitude of total settlement was 57.97 inches, or 4.83 feet, and was calculated at Station 7+50 along cross section SN-3. The post settlement slope from this location, which is near the center of the cell, to the sump, where no settlement was assumed, was 2.0%. A sample calculation showing the settlement calculation procedure and post settlement slope calculation at this location is provided in Attachment E.

The maximum differential settlement between points along a station was calculated to be 13.62 inches at Station 7+13 along cross section SN-4. This differential settlement resulted in a maximum strain of 0.63%.

X. CONCLUSIONS

Given the assumptions and scenarios discussed in the preceding sections, the settlement analysis shows that the post settlement slope near the proposed leachate collection pipes will be at least 1% post settlement, indicating positive drainage and collection of leachate.

In addition, the post settlement slope from the location of greatest settlement to the sump (assuming no settlement at the sump) was still 2%, indicating positive drainage along the cell floor will be maintained. The maximum differential settlement between any two points is within the expected strain tolerances for HDPE and GCL components.

XI. REFERENCES

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Schmertmann, J.M. "The Undisturbed Consolidation of Clay." Transactions of the American Society of Civil Engineers, Vol. 120 (1955): 1201-1233.

Terzaghi, Karl, Ralph B. Peck and Gholamreza Mesri. Soil Mechanics in Engineering Practice, 3rd Edition. New York: John Wiley & Sons, Inc., 1996.

XII. ATTACHMENTS

Attachment A: Plan View
Attachment B: Cross Sections SN 1 – SN 4
Attachment C: Cross Sections WE 1 – WE 3
Attachment D: Settlement Spreadsheet Calculations
Attachment E: Settlement Example Calculation at Station 7+50, Cross Section SN-3

Attachment A Plan View

Settlement Analysis

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NOTE:
FINAL PROPOSED CAP GRADES AND EXISTING SITE GRADES ARE SHOWN IN PLAN VIEW.

DRAWN BY: SD	DATE 03/29/2018
CHECKED BY: MW	JOB NO.: 60398526
SCALE AS SHOWN	

PLAN VIEW
SETTLEMENT CALCULATIONS

PROPOSED EAST PHASE DISPOSAL FACILITY

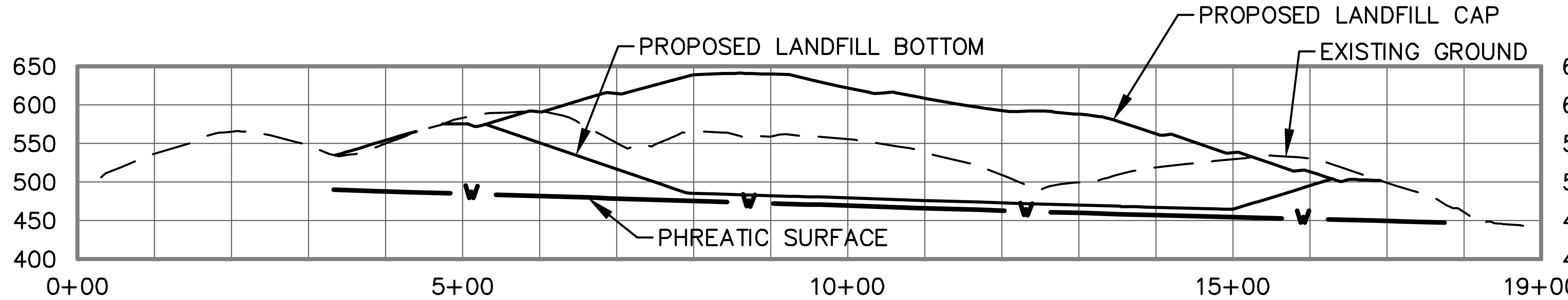
SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

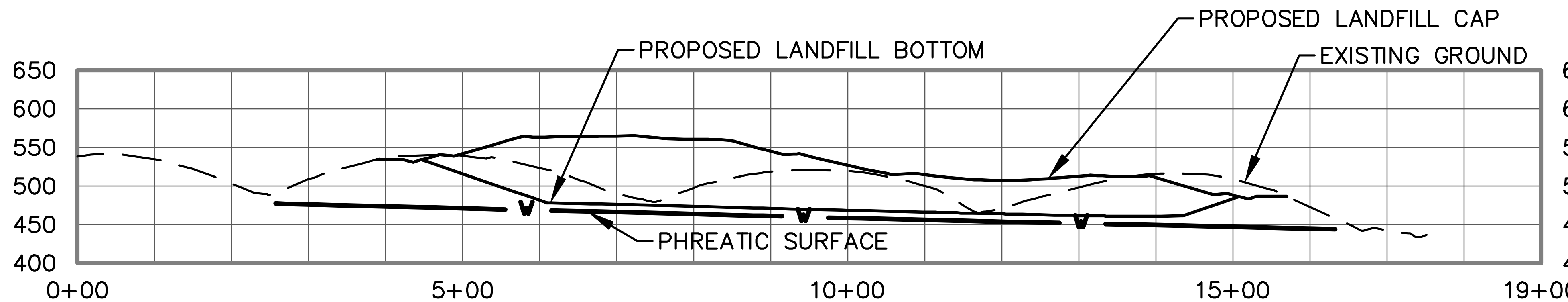
ATTACHMENT A

Attachment B Cross Sections SN 1 – SN 4

Settlement Analysis



XSEC SN 1



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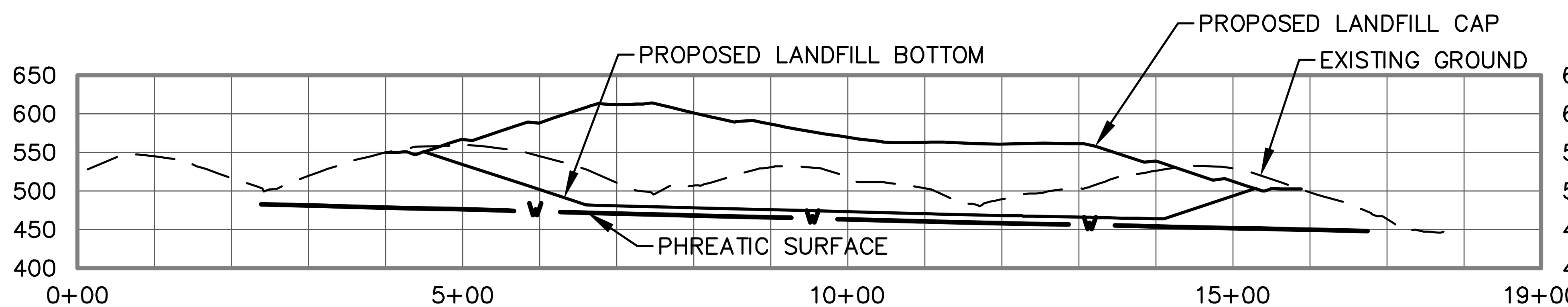

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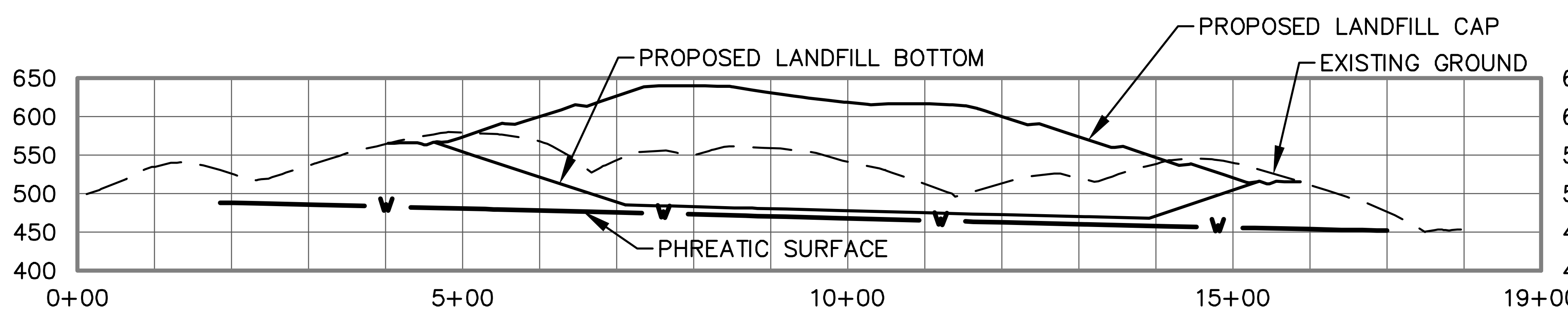

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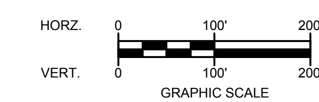

DRAWN BY: SD	DATE 03/29/2018
CHECKED BY: MW	JOB NO.: 60398526
SCALE AS SHOWN	

SOUTH-NORTH (SN)
CROSS SECTIONS
SN 1 TO SN 4

PROPOSED EAST
PHASE DISPOSAL
FACILITY

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

ATTACHMENT B

Attachment C Cross Sections WE 1 – WE 3

Settlement Analysis

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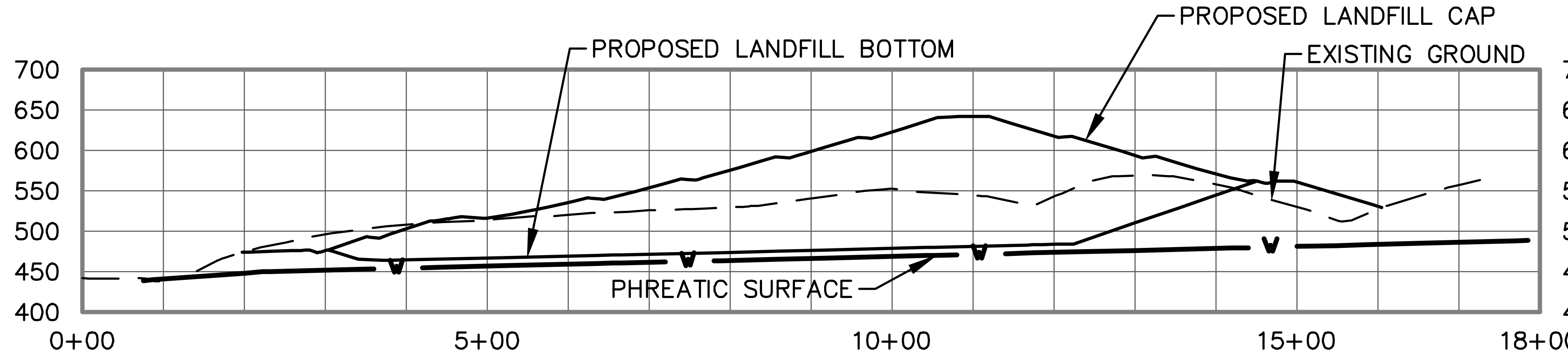
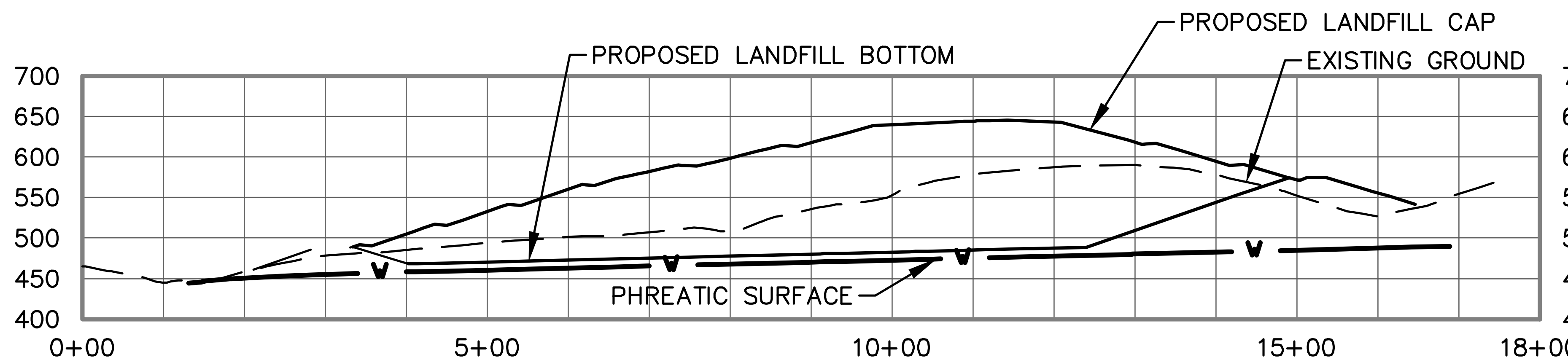


XSEC WE 1



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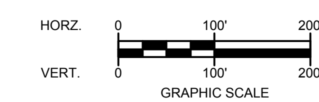

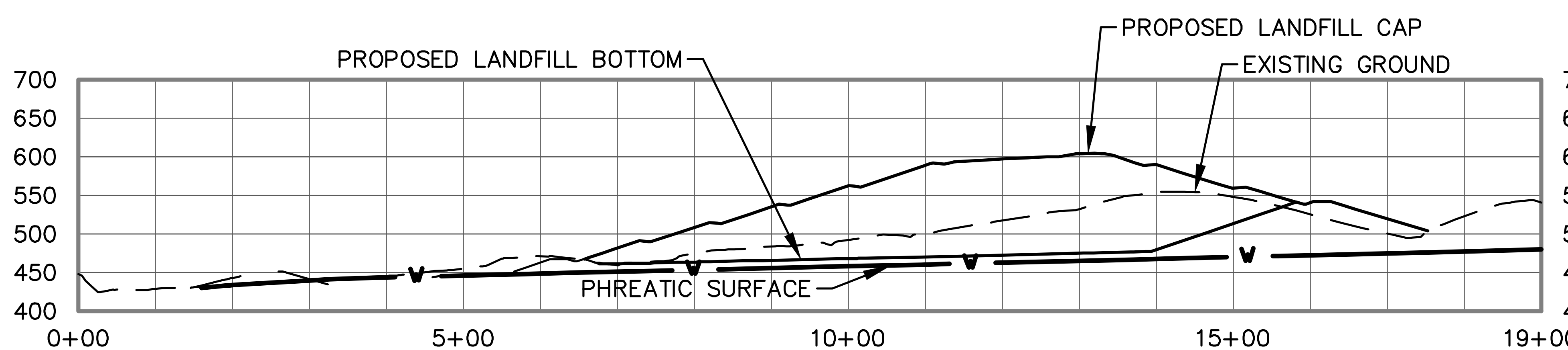

XSEC WE 3

DRAWN BY: SD	DATE 03/29/2018
CHECKED BY: MW	JOB NO.: 60398526
SCALE AS SHOWN	

WEST-EAST (WE)
CROSS SECTIONS
WE 1 TO WE 3

PROPOSED EAST
PHASE DISPOSAL
FACILITY

SCEPTER, INC.
WAVERLY, TENNESSEE
CLASS II DISPOSAL FACILITY

AECOM
1000 CORPORATE CENTRE DR, STE 250
Franklin, TN 37067-6209

ATTACHMENT C

Attachment D Settlement Spreadsheet Calculations

Settlement Analysis

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XSEC SN 1	Foundation Soil				Groundwater		Vertical stress after excavation	Embankment Fill - Waste		
Cross Section Station	Top of ground elevation (ft)	Approximate top of rock elevation (ft)	Height of Soil Column, H (ft)	Depth from ground to midpoint soil column, z (ft)	Water table elevation (ft)	Pore water pressure, u (psf)	Overburden stress, s'_z (psf)	Top of fill elevation (ft)	Bottom of fill elevation (ft)	Assumed embankment height, H_{emb} (ft)
5+33.82	574.78	459.42	115.36	57.68	483.95	0.00	6921.60	682.08	574.78	107.30
5+83.82	557.76	461.78	95.98	47.99	482.41	0.00	5758.80	682.08	557.76	124.32
6+33.82	540.67	455.37	85.30	42.65	480.88	0.00	5118.00	682.08	540.67	141.41
6+83.82	523.54	429.04	94.50	47.25	479.33	189.70	5480.30	682.08	523.51	158.57
7+33.82	506.42	418.08	88.34	44.17	477.77	968.45	4331.95	682.08	506.42	175.66
7+83.82	489.29	433.55	55.74	27.87	476.21	922.90	2421.50	682.08	489.29	192.79
8+33.82	484.65	434.60	50.05	25.03	474.65	937.56	2065.44	682.08	484.65	197.43
8+83.82	483.09	429.24	53.85	26.93	473.09	1056.12	2174.88	682.08	483.09	198.99
9+33.82	481.54	430.78	50.76	25.38	471.54	959.71	2085.89	682.08	481.54	200.54
9+83.82	479.98	426.89	53.09	26.55	469.98	1032.41	2152.99	682.08	479.98	202.10
10+33.82	478.41	421.53	56.88	28.44	468.41	1150.66	2262.14	682.08	478.41	203.67
10+83.82	476.84	412.99	63.85	31.93	466.84	1368.12	2462.88	682.08	476.84	205.24
11+33.82	475.27	400.11	75.16	37.58	465.27	1720.99	2788.61	682.08	475.27	206.81
11+83.82	473.70	385.39	88.31	44.16	463.70	2131.27	3167.33	682.08	473.70	208.38
12+33.82	472.13	365.32	106.81	53.41	462.13	2708.47	3700.13	682.08	472.13	209.95
12+83.82	470.56	368.24	102.32	51.16	460.56	2568.38	3570.82	682.08	470.56	211.52
13+33.82	469.12	374.71	94.41	47.21	459.12	2321.59	3343.01	682.08	469.12	212.96
13+83.82	467.79	386.79	81.00	40.50	457.79	1903.20	2956.80	682.08	467.79	214.29
14+33.82	466.46	392.85	73.61	36.81	456.45	1672.01	2744.59	682.08	466.46	215.62
14+83.82	465.12	398.29	66.83	33.42	455.11	1460.47	2549.33	682.08	465.12	216.96
15+33.82	473.72	403.23	70.49	35.25	453.83	958.15	3271.25	682.08	473.72	208.36
15+83.82	488.91	402.27	86.64	43.32	452.55	434.30	4764.10	682.08	488.91	193.17
16+33.82	503.03	391.33	111.70	55.85	451.25	253.97	6448.03	682.08	503.03	179.05

Assumed Parameters		
Unit weight of foundation soil, g_s	120.00	pcf
Unit weight of waste, g_{waste}	110.00	pcf
Unit weight of water, g_w	62.40	pcf
Foundation soil in situ void ratio, e_0	0.90	
Foundation soil compression index, C_c	0.20	
Foundation soil swell index, C_r	0.04	
Preconsolidation pressure, s'_c	2600.00	psf

XSEC SN 1	Embankment Load - Vertical stress increase																		
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	d	a_1 (rad)	a_2 (rad)	a_3 (rad)	a_4 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)	Ds' (psf)
5+33.82	5+33.82	9+14.45	9+80.59	13+72.05	446.77	391.46	380.63	---	0.06	1.44	1.42	---	---	---	5899.33	-5336.47	---	562.86
5+83.82	4+73.44	9+14.45	9+80.59	14+34.16	396.77	453.57	330.63	110.38	0.06	1.45	1.43	1.16	93.20	31.12	6835.86	-4655.62	1264.72	3444.96
6+33.82	4+12.81	9+14.45	9+80.59	14+96.51	346.77	515.92	280.63	221.01	0.07	1.45	1.42	1.38	79.11	62.30	7775.84	-3933.27	3010.65	6853.22
6+83.82	3+51.92	9+14.45	9+80.59	15+59.04	296.77	578.45	230.63	331.90	0.10	1.41	1.37	1.43	64.99	93.58	8718.01	-3114.61	4683.55	10286.95
7+33.82	2+91.30	9+14.45	9+80.59	16+21.48	246.77	640.89	180.63	442.52	0.13	1.39	1.33	1.47	50.92	124.74	9657.21	-2373.01	6426.18	13710.38
7+83.82	2+30.55	9+14.45	9+80.59	16+83.97	196.77	703.38	130.63	553.27	0.11	1.43	1.36	1.52	36.82	155.97	10601.77	-1754.11	8303.49	17151.15
8+33.82	2+14.08	9+14.45	9+80.59	17+00.90	146.77	720.31	80.63	619.74	0.14	1.40	1.27	1.53	22.73	174.70	10856.43	-1010.64	9361.63	19207.42
8+83.82	2+08.56	9+14.45	9+80.59	17+06.57	96.77	725.98	30.63	675.26	0.24	1.30	0.85	1.53	8.63	190.36	10938.17	-256.75	10204.17	20885.59
9+33.82	2+03.04	9+14.45	9+80.59	17+12.25	46.77	731.66	19.37	711.41	0.46	1.07	0.88	0.65	---	---	11009.38	10954.98	---	21964.35
9+83.82	1+97.52	9+14.45	9+80.59	17+17.93	69.37	716.93	3.23	734.11	0.33	1.21	0.12	1.53	0.89	201.21	11103.83	-3.77	10811.91	21911.97
10+33.82	1+91.96	9+14.45	9+80.59	17+23.65	119.37	722.49	53.23	689.83	0.20	1.34	1.08	1.53	14.59	189.08	11196.82	-551.77	10126.61	20771.65
10+83.82	1+86.39	9+14.45	9+80.59	17+29.38	169.37	728.06	103.23	645.56	0.15	1.38	1.27	1.52	28.29	176.95	11284.68	-1258.85	9426.10	19451.93
11+33.82	1+80.82	9+14.45	9+80.59	17+35.11	219.37	733.63	153.23	601.29	0.13	1.40	1.33	1.51	42.00	164.81	11371.18	-1956.32	8704.36	18119.22
11+83.82	1+75.24	9+14.45	9+80.59	17+40.85	269.37	739.21	203.23	557.03	0.12	1.41	1.36	1.49	55.70	152.68	11457.34	-2646.25	7974.52	16785.60
12+33.82	1+69.67	9+14.45	9+80.59	17+46.58	319.37	744.78	253.23	512.76	0.12	1.41	1.36	1.47	69.41	140.54	11542.86	-3312.41	7219.02	15449.47
12+83.82	1+64.08	9+14.45	9+80.59	17+52.32	369.37	750.37	303.23	468.50	0.09	1.43	1.40	1.46	83.11	128.41	11630.76	-4084.66	6573.51	14119.61
13+33.82	1+58.99	9+14.45	9+80.59	17+57.55	419.37	755.46	353.23	423.73	0.07	1.46	1.44	1.46	96.82	116.14	11711.10	-4874.73	5936.53	12772.90
13+83.82	1+54.27	9+14.45	9+80.59	17+62.42	469.37	760.18	403.23	378.60	0.05	1.48	1.47	1.46	110.52	103.77	11785.11	-5691.22	5320.15	11414.03
14+33.82	1+49.56	9+14.45	9+80.59	17+67.26	519.37	764.89	453.23	333.44	0.04	1.50	1.49	1.46	124.23	91.39	11858.59	-6480.19	4674.67	10053.07
14+83.82	1+44.81	9+14.45	9+80.59	17+72.14	569.37	769.64	503.23	288.32	0.03	1.51	1.50	1.46	137.93	79.03	11932.49	-7265.94	4027.37	8693.93
15+33.82	1+75.29	9+14.45	9+80.59	17+40.79	619.37	739.16	553.23	206.97	0.03	1.51	1.51	1.40	151.64	56.72	11459.50	-8002.40	2784.62	6241.72
15+83.82	2+29.21	9+14.45	9+80.59	16+85.34	669.37	685.24	603.23	101.52	0.03	1.51	1.50	1.17	165.34	27.83	10623.90	-8678.67	1137.64	3082.87
16+33.82	2+79.30	9+14.45	9+80.59	16+33.82	719.37	635.15	653.23	---	0.04	1.49	1.49	---	---	---	9846.96	-9313.04	---	533.92

XSEC SN 1	Primary Consolidation - Settlement (in)	Differential Settlement (in)	Top of ground post settlement (ft)	Distance between top of ground (ft)		Strain	Pre-settlement slope	Post settlement slope
Cross Section Station	overconsolidated clays	overconsolidated clays	overconsolidated clays	original distance between	final distance between (overconsolidated clays)	overconsolidated clays	Slope at top of native soil	overconsolidated clays
5+33.82	27.26	---	572.51	---	---	---	---	---
5+83.82	29.09	1.83	555.34	52.82	52.87	0.0936%	-34.04%	-34.35%
6+33.82	32.56	3.47	537.96	52.84	52.93	0.1781%	-34.18%	-34.76%
6+83.82	42.85	10.30	519.97	52.85	53.14	0.5379%	-34.26%	-35.98%
7+33.82	44.47	1.61	502.71	52.85	52.89	0.0826%	-34.24%	-34.51%
7+83.82	31.08	13.39	486.70	52.85	52.50	0.6640%	-34.26%	-32.03%
8+33.82	29.49	1.59	482.19	50.21	50.20	0.0241%	-9.28%	-9.01%
8+83.82	32.77	3.28	480.36	50.02	50.03	0.0185%	-3.12%	-3.67%
9+33.82	31.59	1.18	478.91	50.02	50.02	0.0059%	-3.10%	-2.90%
9+83.82	32.95	1.37	477.23	50.02	50.03	0.0074%	-3.12%	-3.35%
10+33.82	34.47	1.51	475.54	50.02	50.03	0.0082%	-3.14%	-3.39%
10+83.82	37.52	3.05	473.71	50.02	50.03	0.0173%	-3.14%	-3.65%
11+33.82	42.69	5.16	471.71	50.02	50.04	0.0307%	-3.14%	-4.00%
11+83.82	48.41	5.72	469.67	50.02	50.04	0.0344%	-3.14%	-4.09%
12+33.82	56.43	8.03	467.43	50.02	50.05	0.0509%	-3.14%	-4.48%
12+83.82	52.03	4.40	466.22	50.02	50.01	0.0203%	-3.14%	-2.41%
13+33.82	45.94	6.10	465.29	50.02	50.01	0.0241%	-2.88%	-1.86%
13+83.82	37.41	8.53	464.67	50.02	50.00	0.0277%	-2.66%	-1.24%
14+33.82	31.96	5.45	463.80	50.02	50.01	0.0200%	-2.66%	-1.75%
14+83.82	26.91	5.05	462.88	50.02	50.01	0.0190%	-2.68%	-1.84%
15+33.82	24.19	2.72	471.70	50.73	50.77	0.0767%	17.20%	17.65%
15+83.82	23.37	0.82	486.96	52.26	52.28	0.0381%	30.38%	30.52%
16+33.82	24.70	1.33	500.97	51.96	51.93	0.0576%	28.24%	28.02%

XSEC SN 2	Foundation Soil				Groundwater		Vertical stress after excavation	Embankment Fill - Waste				
Cross Section Station	Top of ground elevation (ft)	Approximate top of rock elevation (ft)	Height of Soil Column, H (ft)	Depth from ground to midpoint soil column, z (ft)	Water table elevation (ft)	Pore water pressure, u (psf)	Overburden stress, s'_z (psf)	Top of fill elevation (ft)	Top of grade break elevation (ft)	Bottom of fill elevation (ft)	Assumed embankment height, H_{emb1} (ft)	Assumed embankment height, H_{emb2} (ft)
4+46.42	533.62	409.52	124.10	62.05	472.31	46.18	7399.82	564.36	---	533.62	30.74	---
4+96.42	516.32	409.39	106.93	53.47	471.02	509.50	5906.30	564.36	---	516.32	48.04	---
5+46.42	499.13	406.06	93.07	46.54	469.72	1068.60	4515.60	564.36	514.82	499.13	49.54	15.69
5+96.42	482.15	394.18	87.97	43.99	468.46	1890.41	3387.79	564.36	514.82	482.15	49.54	32.67
6+46.42	477.18	379.59	97.59	48.80	467.18	2420.81	3434.59	564.36	514.82	477.18	49.54	37.64
6+96.42	475.9	361.98	113.92	56.96	465.90	2930.30	3904.90	564.36	514.82	475.90	49.54	38.92
7+46.42	474.61	349.52	125.09	62.55	464.61	3278.81	4226.59	564.36	514.82	474.61	49.54	40.21
7+96.42	473.33	366.17	107.16	53.58	463.33	2719.39	3710.21	564.36	514.82	473.33	49.54	41.49
8+46.42	472.05	379.2	92.85	46.43	462.05	2272.92	3298.08	564.36	514.82	472.05	49.54	42.77
8+96.42	470.76	387.86	82.90	41.45	460.76	1962.48	3011.52	564.36	514.82	470.76	49.54	44.06
9+46.42	469.51	390.4	79.11	39.56	459.52	1844.86	2901.74	564.36	514.82	469.51	49.54	45.31
9+96.42	468.32	389.66	78.66	39.33	458.34	1831.44	2888.16	564.36	514.82	468.32	49.54	46.50
10+46.42	467.17	382.85	84.32	42.16	457.16	2006.16	3053.04	564.36	514.82	467.17	49.54	47.65
10+96.42	465.99	370.9	95.09	47.55	455.98	2342.18	3363.22	564.36	514.82	465.99	49.54	48.83
11+46.42	464.88	346.76	118.12	59.06	454.88	3061.34	4025.86	564.36	514.82	464.88	49.54	49.94
11+96.42	463.74	339.62	124.12	62.06	453.74	3248.54	4198.66	564.36	514.82	463.74	49.54	51.08
12+46.42	462.65	354.51	108.14	54.07	452.65	2749.97	3738.43	564.36	514.82	462.65	49.54	52.17
12+96.42	461.57	366.78	94.79	47.40	451.57	2333.45	3353.95	564.36	514.82	461.57	49.54	53.25
13+46.42	460.49	378.9	81.59	40.80	450.49	1921.61	2973.79	564.36	514.82	460.49	49.54	54.33
13+96.42	459.38	385.61	73.77	36.89	449.38	1677.62	2748.58	564.36	514.82	459.38	49.54	55.44
14+46.42	465.07	385.15	79.92	39.96	448.23	1442.69	3352.51	564.36	514.82	465.07	49.54	49.75
14+96.42	482.17	379.99	102.18	51.09	447.08	998.40	5132.40	564.36	514.82	482.17	49.54	32.65

Assumed Parameters		
Unit weight of foundation soil, g_s	120.00	pcf
Unit weight of waste, g_{waste}	110.00	pcf
Unit weight of water, g_w	62.40	pcf
Foundation soil in situ void ratio, e_0	0.90	
Foundation soil compression index, C_c	0.20	
Foundation soil swell index, C_r	0.04	
Preconsolidation pressure, s'_c	2600.00	psf

XSEC SN 2	Embankment 1 Load - Vertical stress increase																
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	d	a_1 (rad)	a_2 (rad)	a_3 (rad)	a_4 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)
4+46.42	4+46.42	5+79.51	8+46.42	9+76.54	400.00	130.12	133.09	---	0.04	1.42	1.13	---	---	---	1688.97	-1221.13	---
4+96.42	3+71.49	5+79.51	8+46.42	10+50.66	350.00	204.24	83.09	124.93	0.06	1.42	1.00	1.17	19.19	28.85	2640.18	-671.27	1178.27
5+46.42	3+65.00	5+79.51	8+46.42	10+57.08	300.00	210.66	33.09	181.42	0.06	1.42	0.62	1.32	7.64	41.90	2722.73	-165.35	1936.13
5+96.42	3+65.00	5+79.51	8+46.42	10+57.08	250.00	210.66	16.91	214.51	0.08	1.40	1.02	0.37	---	---	2722.13	2537.82	---
6+46.42	3+65.00	5+79.51	8+46.42	10+57.08	200.00	210.66	66.91	214.51	0.12	1.33	0.46	0.94	---	---	2718.88	2674.93	---
6+96.42	3+65.00	5+79.51	8+46.42	10+57.08	150.00	210.66	116.91	214.51	0.21	1.21	0.28	1.12	---	---	2707.77	2697.16	---
7+46.42	3+65.00	5+79.51	8+46.42	10+57.08	100.00	210.66	166.91	214.51	0.36	1.01	0.20	1.21	---	---	2676.71	2707.30	---
7+96.42	3+65.00	5+79.51	8+46.42	10+57.08	50.00	210.66	216.91	214.51	0.62	0.75	0.12	1.33	---	---	2627.15	2718.40	---
8+46.42	3+65.00	5+79.51	8+46.42	10+57.08	266.91	214.51	210.66	---	0.08	1.40	1.35	---	---	---	2722.14	2348.45	---
8+96.42	3+65.00	5+79.51	8+46.42	10+57.08	316.91	214.51	50.00	160.66	0.05	1.44	0.88	1.32	11.76	37.78	2723.49	-361.79	1743.90
9+46.42	3+65.00	5+79.51	8+46.42	10+57.08	366.91	214.51	100.00	110.66	0.04	1.46	1.19	1.23	23.51	26.03	2723.96	-982.99	1118.77
9+96.42	3+65.00	5+79.51	8+46.42	10+57.08	416.91	214.51	150.00	60.66	0.03	1.48	1.31	1.00	35.27	14.27	2724.17	-1623.18	497.44
10+46.42	3+65.00	5+79.51	8+46.42	10+57.08	466.91	214.51	200.00	10.66	0.03	1.48	1.36	0.25	47.03	2.51	2724.21	-2244.53	21.77
10+96.42	3+65.00	5+79.51	8+46.42	10+57.08	516.91	214.51	39.34	210.66	0.03	1.48	0.69	0.69	---	---	2724.16	-2622.74	---
11+46.42	3+65.00	5+79.51	8+46.42	10+57.08	566.91	214.51	89.34	210.66	0.03	1.47	0.39	0.99	---	---	2723.89	-2674.23	---
11+96.42	3+65.00	5+79.51	8+46.42	10+57.08	616.91	214.51	139.34	210.66	0.03	1.47	0.24	1.15	---	---	2723.95	-2699.69	---
12+46.42	3+65.00	5+79.51	8+46.42	10+57.08	666.91	214.51	189.34	210.66	0.02	1.49	0.14	1.29	---	---	2724.29	-2715.84	---
12+96.42	3+65.00	5+79.51	8+46.42	10+57.08	716.91	214.51	239.34	210.66	0.02	1.50	0.09	1.38	---	---	2724.47	-2721.15	---
13+46.42	3+65.00	5+79.51	8+46.42	10+57.08	766.91	214.51	289.34	210.66	0.01	1.52	0.06	1.43	---	---	2724.58	-2723.24	---
13+96.42	3+65.00	5+79.51	8+46.42	10+57.08	816.91	214.51	339.34	210.66	0.01	1.53	0.04	1.46	---	---	2724.62	-2723.97	---
14+46.42	3+65.00	5+79.51	8+46.42	10+57.08	866.91	214.51	389.34	210.66	0.01	1.52	0.04	1.47	---	---	2724.62	-2724.04	---
14+96.42	3+65.00	5+79.51	8+46.42	10+57.08	916.91	214.51	439.34	210.66	0.01	1.52	0.04	1.46	---	---	2724.55	-2723.68	---

XSEC SN 2	Embankment 2 Load - Vertical stress increase																Total vertical stress increase
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	d	a_1 (rad)	a_2 (rad)	a_3 (rad)	a_4 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)	Ds' (psf)
4+46.42	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	467.83
4+96.42	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	3147.18
5+46.42	2+97.04	3+65.00	13+91.93	14+54.89	845.51	62.96	181.42	67.96	0.00	1.52	0.07	1.32	---	---	862.90	859.29	---	6215.68
5+96.42	2+23.58	3+65.00	13+91.93	15+22.94	795.51	131.01	231.42	141.42	0.01	1.52	0.07	1.38	---	---	1796.75	1794.29	---	8850.98
6+46.42	2+01.98	3+65.00	13+91.93	15+42.95	745.51	151.02	281.42	163.02	0.01	1.51	0.06	1.40	---	---	2070.01	2067.89	---	9531.71
6+96.42	1+96.43	3+65.00	13+91.93	15+48.09	695.51	156.16	331.42	168.57	0.01	1.49	0.06	1.40	---	---	2140.23	2138.12	---	9683.28
7+46.42	1+90.87	3+65.00	13+91.93	15+53.24	645.51	161.31	381.42	174.13	0.02	1.47	0.05	1.41	---	---	2210.94	2209.21	---	9804.17
7+96.42	1+85.32	3+65.00	13+91.93	15+58.38	595.51	166.45	431.42	179.68	0.02	1.48	0.04	1.45	---	---	2281.46	2280.85	---	9907.87
8+46.42	1+79.76	3+65.00	13+91.93	15+63.54	545.51	171.61	481.42	185.24	0.02	1.49	0.03	1.47	---	---	2351.94	2351.80	---	9774.32
8+96.42	1+74.19	3+65.00	13+91.93	15+68.69	495.51	176.76	531.42	190.81	0.02	1.49	0.02	1.49	---	---	2422.92	2422.99	---	8951.50
9+46.42	1+68.79	3+65.00	13+91.93	15+73.70	445.51	181.77	581.42	196.21	0.03	1.48	0.02	1.50	---	---	2491.60	2491.83	---	7843.18
9+96.42	1+63.64	3+65.00	13+91.93	15+78.47	395.51	186.54	631.42	201.36	0.03	1.47	0.02	1.51	---	---	2556.90	2557.33	---	6712.66
10+46.42	1+58.63	3+65.00	13+91.93	15+83.11	345.51	191.18	681.42	206.37	0.04	1.45	0.01	1.51	---	---	2619.69	2620.57	---	5741.71
10+96.42	1+53.54	3+65.00	13+91.93	15+87.83	295.51	195.90	731.42	211.46	0.06	1.41	0.01	1.51	---	---	2683.41	2685.44	---	5470.27
11+46.42	1+48.74	3+65.00	13+91.93	15+92.27	245.51	200.34	781.42	216.26	0.10	1.33	0.02	1.50	---	---	2740.07	2746.35	---	5536.08
11+96.42	1+43.77	3+65.00	13+91.93	15+96.87	195.51	204.94	831.42	221.23	0.15	1.26	0.02	1.50	---	---	2796.51	2809.05	---	5629.81
12+46.42	1+39.05	3+65.00	13+91.93	16+01.25	145.51	209.32	881.42	225.95	0.20	1.22	0.01	1.51	---	---	2852.87	2869.15	---	5730.48
12+96.42	1+34.39	3+65.00	13+91.93	16+05.57	95.51	213.64	931.42	230.61	0.31	1.11	0.01	1.52	---	---	2902.27	2928.63	---	5834.23
13+46.42	1+29.69	3+65.00	13+91.93	16+09.92	45.51	217.99	981.42	235.31	0.58	0.84	0.01	1.53	---	---	2925.20	2988.08	---	5914.62
13+96.42	1+24.90	3+65.00	13+91.93	16+14.36	1031.42	240.10	4.49	217.94	0.01	1.54	0.12	1.40	1.12	54.32	3049.16	-4.75	2668.73	5713.79
14+46.42	2+49.38	3+65.00	13+91.93	15+91.53	1081.42	115.62	54.49	145.11	0.00	1.53	0.94	1.30	13.58	36.17	2736.20	-446.03	1649.03	3939.78
14+96.42	2+23.58	3+65.00	13+91.93	15+22.94	1131.42	141.42	104.49	26.52	0.01	1.53	1.12	0.48	26.05	6.60	1795.69	-1017.95	110.65	889.26

XSEC SN 2	Primary Consolidation - Settlement (in)	Differential Settlement (in)	Top of ground post settlement (ft)	Distance between top of ground (ft)			Strain	Pre-settlement slope	Post settlement slope
Cross Section Station	overconsolidated clays	overconsolidated clays	overconsolidated clays	original distance between	final distance between (normally consolidated clays)	final distance between (overconsolidated clays)	overconsolidated clays	Slope at top of native soil	overconsolidated clays
4+46.42	30.57	---	531.07	---	---	---	---	---	---
4+96.42	31.78	1.21	513.67	52.91	53.20	52.94	0.0625%	-34.60%	-34.80%
5+46.42	33.37	1.59	496.35	52.87	53.14	52.92	0.0818%	-34.38%	-34.65%
5+96.42	36.10	2.73	479.14	52.80	53.05	52.88	0.1394%	-33.96%	-34.41%
6+46.42	41.52	5.42	473.72	50.25	50.29	50.29	0.0929%	-9.94%	-10.84%
6+96.42	49.13	7.61	471.81	50.02	50.02	50.04	0.0405%	-2.56%	-3.83%
7+46.42	54.51	5.37	470.07	50.02	50.02	50.03	0.0271%	-2.58%	-3.48%
7+96.42	46.58	7.92	469.45	50.02	50.01	50.00	0.0251%	-2.56%	-1.24%
8+46.42	39.92	6.66	468.72	50.02	50.01	50.01	0.0222%	-2.56%	-1.45%
8+96.42	34.04	5.88	467.92	50.02	50.01	50.01	0.0205%	-2.58%	-1.60%
9+46.42	30.31	3.73	466.98	50.02	50.01	50.01	0.0136%	-2.50%	-1.88%
9+96.42	27.73	2.58	466.01	50.01	50.01	50.01	0.0093%	-2.38%	-1.95%
10+46.42	27.44	0.29	464.88	50.01	50.01	50.01	0.0011%	-2.30%	-2.25%
10+96.42	30.56	3.11	463.44	50.01	50.02	50.02	0.0136%	-2.36%	-2.88%
11+46.42	39.36	8.80	461.60	50.01	50.02	50.03	0.0433%	-2.22%	-3.69%
11+96.42	42.01	2.65	460.24	50.01	50.01	50.02	0.0110%	-2.28%	-2.72%
12+46.42	36.18	5.83	459.63	50.01	50.01	50.00	0.0164%	-2.18%	-1.21%
12+96.42	31.50	4.69	458.95	50.01	50.01	50.00	0.0138%	-2.16%	-1.38%
13+46.42	26.91	4.59	458.25	50.01	50.01	50.00	0.0136%	-2.16%	-1.39%
13+96.42	23.65	3.25	457.41	50.01	50.01	50.01	0.0106%	-2.22%	-1.68%
14+46.42	21.49	2.16	463.28	50.32	50.38	50.34	0.0411%	11.38%	11.74%
14+96.42	19.73	1.77	480.53	52.84	53.19	52.89	0.0905%	34.20%	34.49%

XSEC SN 3	Foundation Soil				Groundwater		Vertical stress after excavation	Embankment Fill - Waste				
Cross Section Station	Top of ground elevation (ft)	Approximate top of rock elevation (ft)	Height of Soil Column, H (ft)	Depth from ground to midpoint soil column, z (ft)	Water table elevation (ft)	Pore water pressure, u (psf)	Overburden stress, s'_z (psf)	Top of fill elevation (ft)	Top of grade break elevation (ft)	Bottom of fill elevation (ft)	Assumed embankment height, H_{emb1} (ft)	Assumed embankment height, H_{emb2} (ft)
4+50.00	550.46	427.63	122.83	61.42	477.13	0.00	7369.80	613.71	562.29	550.46	51.42	11.83
5+00.00	534.09	429.4	104.69	52.35	475.85	0.00	6281.40	613.71	562.29	534.09	51.42	28.20
5+50.00	517.73	424.87	92.86	46.43	474.56	203.42	5368.18	613.71	562.29	517.73	51.42	44.56
6+00.00	501.37	415.14	86.23	43.12	473.28	937.56	4236.24	613.71	562.29	501.37	51.42	60.92
6+50.00	485	401.25	83.75	41.88	472.00	1801.80	3223.20	613.71	562.29	485.00	51.42	77.29
7+00.00	480.72	380.37	100.35	50.18	470.72	2506.92	3514.08	613.71	562.29	480.72	51.42	81.57
7+50.00	479.44	366.22	113.22	56.61	469.44	2908.46	3884.74	613.71	562.29	479.44	51.42	82.85
8+00.00	478.16	376.77	101.39	50.70	468.16	2539.37	3544.03	613.71	562.29	478.16	51.42	84.13
8+50.00	476.87	389.22	87.65	43.83	466.87	2110.68	3148.32	613.71	562.29	476.87	51.42	85.42
9+00.00	475.59	400.87	74.72	37.36	465.59	1707.26	2775.94	613.71	562.29	475.59	51.42	86.70
9+50.00	474.3	400.45	73.85	36.93	464.30	1680.12	2750.88	613.71	562.29	474.30	51.42	87.99
10+00.00	473.02	386.76	86.26	43.13	463.02	2067.31	3108.29	613.71	562.29	473.02	51.42	89.27
10+50.00	471.73	380.88	90.85	45.43	461.73	2210.52	3240.48	613.71	562.29	471.73	51.42	90.56
11+00.00	470.44	373.19	97.25	48.63	460.44	2410.20	3424.80	613.71	562.29	470.44	51.42	91.85
11+50.00	469.25	355.49	113.76	56.88	459.25	2925.31	3900.29	613.71	562.29	469.25	51.42	93.04
12+00.00	468.11	359.09	109.02	54.51	458.11	2777.42	3763.78	613.71	562.29	468.11	51.42	94.18
12+50.00	467.03	367.48	99.55	49.78	457.03	2481.96	3491.04	613.71	562.29	467.03	51.42	95.26
13+00.00	465.91	373.15	92.76	46.38	455.91	2270.11	3295.49	613.71	562.29	465.91	51.42	96.38
13+50.00	464.85	387.4	77.45	38.73	454.85	1792.44	2854.56	613.71	562.29	464.85	51.42	97.44
14+00.00	463.78	396.13	67.65	33.83	453.78	1486.68	2572.32	613.71	562.29	463.78	51.42	98.51
14+50.00	476.7	402.51	74.19	37.10	452.72	818.38	3633.02	613.71	562.29	476.70	51.42	85.59
15+00.00	493.47	399.2	94.27	47.14	451.70	334.78	5321.42	613.71	562.29	493.47	51.42	68.82

Assumed Parameters		
Unit weight of foundation soil, g_s	120.00	pcf
Unit weight of waste, g_{waste}	110.00	pcf
Unit weight of water, g_w	62.40	pcf
Foundation soil in situ void ratio, e_0	0.90	
Foundation soil compression index, C_c	0.20	
Foundation soil swell index, C_r	0.04	
Preconsolidation pressure, s'_c	2600.00	psf

XSEC SN 3	Embankment 1 Load - Vertical stress increase																
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	d	a_1 (rad)	a_2 (rad)	a_3 (rad)	a_4 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)
4+50.00	4+91.99	6+75.78	7+46.82	10+57.27	296.82	310.45	41.99	183.79	0.10	1.37	0.71	0.60	---	---	2824.35	-2640.13	---
5+00.00	4+91.99	6+75.78	7+46.82	10+57.27	246.82	310.45	175.78	8.01	0.12	1.36	1.28	0.15	49.18	2.24	2824.56	-2206.51	11.91
5+50.00	4+91.99	6+75.78	7+46.82	10+57.27	196.82	310.45	125.78	58.01	0.14	1.34	1.22	0.90	35.19	16.23	2824.01	-1499.74	509.08
6+00.00	4+91.99	6+75.78	7+46.82	10+57.27	146.82	310.45	75.78	108.01	0.19	1.29	1.05	1.19	21.20	30.22	2822.00	-782.03	1260.23
6+50.00	4+91.99	6+75.78	7+46.82	10+57.27	96.82	310.45	25.78	158.01	0.31	1.16	0.55	1.31	7.21	44.21	2815.31	-139.31	2030.53
7+00.00	4+91.99	6+75.78	7+46.82	10+57.27	46.82	310.45	24.22	183.79	0.68	0.75	0.88	0.45	---	---	2761.65	2611.78	---
7+50.00	4+91.99	6+75.78	7+46.82	10+57.27	74.22	183.79	3.18	307.27	0.44	0.92	0.06	1.39	0.53	50.89	2755.95	-1.04	2474.31
8+00.00	4+91.99	6+75.78	7+46.82	10+57.27	124.22	183.79	53.18	257.27	0.22	1.18	0.81	1.38	8.81	42.61	2807.41	-249.65	2053.28
8+50.00	4+91.99	6+75.78	7+46.82	10+57.27	174.22	183.79	103.18	207.27	0.12	1.32	1.17	1.36	17.09	34.33	2821.50	-699.60	1637.68
9+00.00	4+91.99	6+75.78	7+46.82	10+57.27	224.22	183.79	153.18	157.27	0.07	1.41	1.33	1.34	25.37	26.05	2825.79	-1182.84	1220.02
9+50.00	4+91.99	6+75.78	7+46.82	10+57.27	274.22	183.79	203.18	107.27	0.05	1.44	1.39	1.24	33.65	17.77	2826.72	-1638.94	771.08
10+00.00	4+91.99	6+75.78	7+46.82	10+57.27	324.22	183.79	253.18	57.27	0.05	1.44	1.40	0.93	41.94	9.48	2826.64	-2058.92	307.14
10+50.00	4+91.99	6+75.78	7+46.82	10+57.27	374.22	183.79	303.18	7.27	0.04	1.45	1.42	0.16	50.22	1.20	2826.91	-2500.59	6.67
11+00.00	4+91.99	6+75.78	7+46.82	10+57.27	424.22	183.79	42.73	310.45	0.03	1.46	0.71	0.72	---	---	2827.04	-2758.46	---
11+50.00	4+91.99	6+75.78	7+46.82	10+57.27	474.22	183.79	92.73	310.45	0.03	1.45	0.41	1.02	---	---	2826.83	-2796.28	---
12+00.00	4+91.99	6+75.78	7+46.82	10+57.27	524.22	183.79	142.73	310.45	0.03	1.47	0.25	1.21	---	---	2827.24	-2815.46	---
12+50.00	4+91.99	6+75.78	7+46.82	10+57.27	574.22	183.79	192.73	310.45	0.02	1.48	0.15	1.32	---	---	2827.58	-2822.86	---
13+00.00	4+91.99	6+75.78	7+46.82	10+57.27	624.22	183.79	242.73	310.45	0.02	1.50	0.11	1.38	---	---	2827.76	-2825.52	---
13+50.00	4+91.99	6+75.78	7+46.82	10+57.27	674.22	183.79	292.73	310.45	0.01	1.51	0.07	1.44	---	---	2827.94	-2827.11	---
14+00.00	4+91.99	6+75.78	7+46.82	10+57.27	724.22	183.79	342.73	310.45	0.01	1.52	0.05	1.47	---	---	2828.01	-2827.64	---
14+50.00	4+91.99	6+75.78	7+46.82	10+57.27	774.22	183.79	392.73	310.45	0.01	1.52	0.04	1.48	---	---	2828.00	-2827.66	---
15+00.00	4+91.99	6+75.78	7+46.82	10+57.27	824.22	183.79	442.73	310.45	0.01	1.51	0.04	1.46	---	---	2827.93	-2827.43	---

XSEC SN 3	Embankment 2 Load - Vertical stress increase																		Total vertical stress increase
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	d	a_1 (rad)	a_2 (rad)	a_3 (rad)	a_4 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)		Ds' (psf)
4+50.00	4+49.71	4+91.99	13+05.87	13+50.17	855.87	44.30	41.99	0.29	0.00	1.50	0.60	0.00	11.75	0.08	650.56	-246.73	0.01		588.06
5+00.00	3+91.22	4+91.99	13+05.87	14+11.44	805.87	105.57	8.01	100.77	0.01	1.51	0.97	0.15	---	---	1550.85	1184.32	---		3365.13
5+50.00	3+32.73	4+91.99	13+05.87	14+72.72	755.87	166.85	58.01	159.26	0.01	1.51	0.46	0.90	---	---	2450.62	2386.27	---		6670.24
6+00.00	2+74.25	4+91.99	13+05.87	15+33.99	705.87	228.12	108.01	217.74	0.01	1.51	0.25	1.19	---	---	3350.39	3332.53	---		9983.12
6+50.00	2+15.76	4+91.99	13+05.87	15+95.27	655.87	289.40	158.01	276.23	0.02	1.51	0.16	1.31	---	---	4250.68	4243.00	---		13200.20
7+00.00	2+00.45	4+91.99	13+05.87	16+11.30	605.87	305.43	208.01	291.54	0.03	1.49	0.14	1.33	---	---	4485.76	4478.78	---		14337.97
7+50.00	1+95.88	4+91.99	13+05.87	16+16.10	555.87	310.23	258.01	296.11	0.04	1.47	0.11	1.35	---	---	4555.68	4550.02	---		14334.92
8+00.00	1+91.29	4+91.99	13+05.87	16+20.90	505.87	315.03	308.01	300.70	0.04	1.47	0.08	1.41	---	---	4626.17	4623.88	---		13861.09
8+50.00	1+86.70	4+91.99	13+05.87	16+25.71	455.87	319.84	358.01	305.29	0.04	1.47	0.06	1.45	---	---	4697.28	4696.60	---		13153.45
9+00.00	1+82.11	4+91.99	13+05.87	16+30.52	405.87	324.65	408.01	309.88	0.04	1.48	0.04	1.48	---	---	4767.82	4767.81	---		12398.59
9+50.00	1+77.51	4+91.99	13+05.87	16+35.34	355.87	329.47	458.01	314.48	0.05	1.47	0.03	1.49	---	---	4838.55	4838.94	---		11636.36
10+00.00	1+72.92	4+91.99	13+05.87	16+40.15	305.87	334.28	508.01	319.07	0.07	1.43	0.03	1.49	---	---	4907.82	4909.22	---		10891.91
10+50.00	1+68.31	4+91.99	13+05.87	16+44.98	255.87	339.11	558.01	323.68	0.10	1.40	0.03	1.49	---	---	4977.24	4980.21	---		10290.45
11+00.00	1+63.70	4+91.99	13+05.87	16+49.81	205.87	343.94	608.01	328.29	0.14	1.34	0.03	1.49	---	---	5044.75	5051.17	---		10164.49
11+50.00	1+59.45	4+91.99	13+05.87	16+54.26	155.87	348.39	658.01	332.54	0.24	1.22	0.03	1.48	---	---	5097.55	5116.43	---		10244.53
12+00.00	1+55.39	4+91.99	13+05.87	16+58.52	105.87	352.65	708.01	336.60	0.36	1.10	0.02	1.49	---	---	5143.28	5179.33	---		10334.39
12+50.00	1+51.51	4+91.99	13+05.87	16+62.58	55.87	356.71	758.01	340.48	0.61	0.84	0.02	1.51	---	---	5156.31	5238.93	---		10399.97
13+00.00	1+47.51	4+91.99	13+05.87	16+66.77	5.87	360.90	808.01	344.48	1.32	0.13	0.02	1.51	---	---	4948.81	5300.65	---		10251.70
13+50.00	1+43.72	4+91.99	13+05.87	16+70.74	858.01	348.27	44.13	320.74	0.01	1.53	0.85	1.45	11.78	85.66	5359.07	-350.82	4350.92		9360.00
14+00.00	1+39.90	4+91.99	13+05.87	16+74.74	908.01	352.09	94.13	274.74	0.01	1.53	1.23	1.45	25.14	73.37	5417.98	-1079.03	3720.65		8059.96
14+50.00	2+32.54	4+91.99	13+05.87	16+26.35	958.01	259.45	144.13	176.35	0.01	1.53	1.32	1.36	38.49	47.10	4707.37	-1777.46	2248.59		5178.83
15+00.00	2+46.01	4+91.99	13+05.87	15+63.57	1008.01	245.98	194.13	63.57	0.01	1.52	1.33	0.93	51.84	16.98	3784.98	-2418.85	554.57		1921.21

XSEC SN 3	Primary Consolidation - Settlement (in)	Differential Settlement (in)	Top of ground post settlement (ft)	Distance between top of ground (ft)		Strain	Pre-settlement slope	Post settlement slope
Cross Section Station	overconsolidated clays	overconsolidated clays	overconsolidated clays	original distance between	final distance between (overconsolidated clays)	overconsolidated clays	Slope at top of native soil	overconsolidated clays
4+50.00	30.67	---	547.90	---	---	---	---	---
5+00.00	32.58	1.91	531.37	52.61	52.66	0.0948%	-32.74%	-33.06%
5+50.00	35.34	2.76	514.78	52.61	52.68	0.1368%	-32.72%	-33.18%
6+00.00	37.88	2.53	498.21	52.61	52.67	0.1256%	-32.72%	-33.14%
6+50.00	41.35	3.48	481.55	52.61	52.70	0.1727%	-32.74%	-33.32%
7+00.00	51.37	10.02	476.44	50.18	50.26	0.1556%	-8.56%	-10.23%
7+50.00	57.97	6.60	474.61	50.02	50.03	0.0342%	-2.56%	-3.66%
8+00.00	51.15	6.82	473.90	50.02	50.01	0.0226%	-2.56%	-1.42%
8+50.00	43.21	7.94	473.27	50.02	50.00	0.0254%	-2.58%	-1.26%
9+00.00	35.89	7.33	472.60	50.02	50.00	0.0238%	-2.56%	-1.34%
9+50.00	34.43	1.46	471.43	50.02	50.01	0.0060%	-2.58%	-2.34%
10+00.00	38.99	4.56	469.77	50.02	50.03	0.0223%	-2.56%	-3.32%
10+50.00	40.01	1.02	468.40	50.02	50.02	0.0045%	-2.58%	-2.75%
11+00.00	42.64	2.64	466.89	50.02	50.02	0.0123%	-2.58%	-3.02%
11+50.00	50.32	7.68	465.06	50.01	50.03	0.0386%	-2.38%	-3.66%
12+00.00	48.34	1.98	464.08	50.01	50.01	0.0070%	-2.28%	-1.95%
12+50.00	44.15	4.19	463.35	50.01	50.01	0.0126%	-2.16%	-1.46%
13+00.00	40.79	3.36	462.51	50.01	50.01	0.0110%	-2.24%	-1.68%
13+50.00	32.47	8.32	462.14	50.01	50.00	0.0198%	-2.12%	-0.73%
14+00.00	26.17	6.30	461.60	50.01	50.00	0.0169%	-2.14%	-1.09%
14+50.00	23.48	2.70	474.74	51.64	51.70	0.1097%	25.84%	26.29%
15+00.00	22.79	0.69	491.57	52.74	52.76	0.0348%	33.54%	33.66%

XSEC SN 4	Foundation Soil				Groundwater		Vertical stress after excavation	Embankment Fill - Waste				
Cross Section Station	Top of ground elevation (ft)	Approximate top of rock elevation (ft)	Height of Soil Column, H (ft)	Depth from ground to midpoint soil column, z (ft)	Water table elevation (ft)	Pore water pressure, u (psf)	Overburden stress, s'_z (psf)	Top of fill elevation (ft)	Top of grade break elevation (ft)	Bottom of fill elevation (ft)	Assumed embankment height, H_{emb1} (ft)	Assumed embankment height, H_{emb2} (ft)
4+63.00	566.51	447.17	119.34	59.67	481.65	0.00	7160.40	639.21	615.68	566.51	23.53	49.17
5+13.00	550.17	448.48	101.69	50.85	480.40	0.00	6101.40	639.21	615.68	550.17	23.53	65.51
5+63.00	533.8	444.65	89.15	44.58	479.13	0.00	5349.00	639.21	615.68	533.80	23.53	81.88
6+13.00	517.44	433.2	84.24	42.12	477.85	157.87	4896.53	639.21	615.68	517.44	23.53	98.24
6+63.00	501.07	400.9	100.17	50.09	476.57	1596.50	4413.70	639.21	615.68	501.07	23.53	114.61
7+13.00	485.29	419.43	65.86	32.93	475.29	1430.83	2520.77	639.21	615.68	485.29	23.53	130.39
7+63.00	484.01	425.98	58.03	29.02	474.01	1186.54	2295.26	639.21	615.68	484.01	23.53	131.67
8+13.00	482.73	423.3	59.43	29.72	472.73	1230.22	2335.58	639.21	615.68	482.73	23.53	132.95
8+63.00	481.46	430.7	50.76	25.38	471.46	959.71	2085.89	639.21	615.68	481.46	23.53	134.22
9+13.00	480.18	428.58	51.60	25.80	470.18	985.92	2110.08	639.21	615.68	480.18	23.53	135.50
9+63.00	478.9	421.89	57.01	28.51	468.90	1154.71	2265.89	639.21	615.68	478.90	23.53	136.78
10+13.00	477.61	408.75	68.86	34.43	467.61	1524.43	2607.17	639.21	615.68	477.61	23.53	138.07
10+63.00	476.32	395.6	80.72	40.36	466.32	1894.46	2948.74	639.21	615.68	476.32	23.53	139.36
11+13.00	475.03	377.89	97.14	48.57	465.03	2406.77	3421.63	639.21	615.68	475.03	23.53	140.65
11+63.00	473.74	372.82	100.92	50.46	463.74	2524.70	3530.50	639.21	615.68	473.74	23.53	141.94
12+13.00	472.45	387.11	85.34	42.67	462.45	2038.61	3081.79	639.21	615.68	472.45	23.53	143.23
12+63.00	471.16	395.41	75.75	37.88	461.16	1739.40	2805.60	639.21	615.68	471.16	23.53	144.52
13+13.00	469.89	387.56	82.33	41.17	459.89	1944.70	2995.10	639.21	615.68	469.89	23.53	145.79
13+63.00	468.81	398.75	70.06	35.03	458.81	1561.87	2641.73	639.21	615.68	468.81	23.53	146.87
14+13.00	475.43	412.41	63.02	31.51	457.73	861.74	2919.46	639.21	615.68	475.43	23.53	140.25
14+63.00	492.15	415.02	77.13	38.57	456.70	194.38	4433.42	639.21	615.68	492.15	23.53	123.53
15+13.00	508.88	407.13	101.75	50.88	455.64	0.00	6105.00	639.21	615.68	508.88	23.53	106.80

Assumed Parameters		
Unit weight of foundation soil, g_s	120.00	pcf
Unit weight of waste, g_{waste}	110.00	pcf
Unit weight of water, g_w	62.40	pcf
Foundation soil in situ void ratio, e_0	0.90	
Foundation soil compression index, C_c	0.20	
Foundation soil swell index, C_r	0.04	
Preconsolidation pressure, s'_c	2600.00	psf

XSEC SN 4	Embankment 1 Load - Vertical stress increase																	
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	d	a_1 (rad)	a_2 (rad)	a_3 (rad)	a_4 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)
4+63.00	6+46.96	7+35.00	8+47.00	10+30.00	384.00	183.00	183.96	88.04	0.05	1.42	0.10	1.26	---	---	1293.00	-1284.42	---
5+13.00	6+46.96	7+35.00	8+47.00	10+30.00	334.00	183.00	133.96	88.04	0.05	1.42	0.14	1.21	---	---	1293.14	-1281.16	---
5+63.00	6+46.96	7+35.00	8+47.00	10+30.00	284.00	183.00	83.96	88.04	0.06	1.42	0.23	1.08	---	---	1293.13	-1269.47	---
6+13.00	6+46.96	7+35.00	8+47.00	10+30.00	234.00	183.00	33.96	88.04	0.08	1.39	0.56	0.68	---	---	1292.78	-1198.17	---
6+63.00	6+46.96	7+35.00	8+47.00	10+30.00	184.00	183.00	72.00	16.04	0.13	1.31	0.96	0.31	19.25	4.28	1290.20	-649.09	46.45
7+13.00	6+46.96	7+35.00	8+47.00	10+30.00	134.00	183.00	22.00	66.04	0.14	1.33	0.59	1.11	5.88	17.65	1291.80	-121.26	684.89
7+63.00	6+46.96	7+35.00	8+47.00	10+30.00	84.00	183.00	28.00	88.04	0.22	1.24	0.56	0.77	---	---	1289.81	1238.54	---
8+13.00	6+46.96	7+35.00	8+47.00	10+30.00	34.00	183.00	78.00	88.04	0.58	0.85	0.19	1.21	---	---	1271.14	1284.67	---
8+63.00	6+46.96	7+35.00	8+47.00	10+30.00	128.00	88.04	16.00	167.00	0.08	1.38	0.56	1.42	2.06	21.47	1292.19	-40.57	1067.47
9+13.00	6+46.96	7+35.00	8+47.00	10+30.00	178.00	88.04	66.00	117.00	0.05	1.43	1.20	1.35	8.49	15.04	1293.23	-356.17	712.90
9+63.00	6+46.96	7+35.00	8+47.00	10+30.00	228.00	88.04	116.00	67.00	0.03	1.45	1.33	1.17	14.92	8.61	1293.49	-694.72	352.28
10+13.00	6+46.96	7+35.00	8+47.00	10+30.00	278.00	88.04	166.00	17.00	0.03	1.45	1.37	0.46	21.35	2.18	1293.46	-1021.37	35.01
10+63.00	6+46.96	7+35.00	8+47.00	10+30.00	328.00	88.04	33.00	183.00	0.03	1.45	0.70	0.69	---	---	1293.44	-1246.06	---
11+13.00	6+46.96	7+35.00	8+47.00	10+30.00	378.00	88.04	83.00	183.00	0.02	1.44	0.35	1.04	---	---	1293.31	-1275.71	---
11+63.00	6+46.96	7+35.00	8+47.00	10+30.00	428.00	88.04	133.00	183.00	0.02	1.45	0.20	1.21	---	---	1293.48	-1286.01	---
12+13.00	6+46.96	7+35.00	8+47.00	10+30.00	478.00	88.04	183.00	183.00	0.01	1.48	0.11	1.34	---	---	1293.85	-1291.64	---
12+63.00	6+46.96	7+35.00	8+47.00	10+30.00	528.00	88.04	233.00	183.00	0.01	1.50	0.07	1.41	---	---	1293.99	-1293.14	---
13+13.00	6+46.96	7+35.00	8+47.00	10+30.00	578.00	88.04	283.00	183.00	0.01	1.50	0.06	1.43	---	---	1293.99	-1293.34	---
13+63.00	6+46.96	7+35.00	8+47.00	10+30.00	628.00	88.04	333.00	183.00	0.01	1.52	0.04	1.47	---	---	1294.07	-1293.81	---
14+13.00	6+46.96	7+35.00	8+47.00	10+30.00	678.00	88.04	383.00	183.00	0.01	1.52	0.03	1.49	---	---	1294.10	-1293.98	---
14+63.00	6+46.96	7+35.00	8+47.00	10+30.00	728.00	88.04	433.00	183.00	0.01	1.52	0.03	1.48	---	---	1294.08	-1293.92	---
15+13.00	6+46.96	7+35.00	8+47.00	10+30.00	778.00	88.04	483.00	183.00	0.01	1.51	0.03	1.47	---	---	1294.02	-1293.75	---

XSEC SN 4	Embankment 2 Load - Vertical stress increase																		Total vertical stress increase
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	d	a_1 (rad)	a_2 (rad)	a_3 (rad)	a_4 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)		Ds' (psf)
4+63.00	4+63.00	6+46.96	11+50.00	13+39.17	687.00	189.17	183.96	---	0.02	1.48	1.26	---	---	---	2703.83	-2164.35	---		548.06
5+13.00	4+01.86	6+46.96	11+50.00	14+02.03	637.00	252.03	133.96	111.14	0.02	1.49	1.21	1.14	35.81	29.70	3602.57	-1514.71	1187.31		3287.16
5+63.00	3+40.64	6+46.96	11+50.00	14+64.98	587.00	314.98	83.96	222.36	0.03	1.50	1.08	1.37	22.44	59.44	4502.95	-850.72	2857.45		6533.34
6+13.00	2+79.42	6+46.96	11+50.00	15+27.93	537.00	377.93	33.96	333.58	0.03	1.49	0.68	1.45	9.08	89.16	5402.69	-215.73	4511.69		9793.26
6+63.00	2+18.20	6+46.96	11+50.00	15+90.88	487.00	440.88	16.04	428.76	0.05	1.47	1.15	0.31	---	---	6302.39	6026.04	---		13015.99
7+13.00	1+59.14	6+46.96	11+50.00	16+51.61	437.00	501.61	66.04	487.82	0.04	1.50	0.40	1.11	---	---	7171.01	7149.51	---		16175.95
7+63.00	1+54.35	6+46.96	11+50.00	16+56.54	387.00	506.54	116.04	492.61	0.04	1.50	0.20	1.33	---	---	7241.45	7236.60	---		17006.40
8+13.00	1+49.58	6+46.96	11+50.00	16+61.45	337.00	511.45	166.04	497.38	0.05	1.48	0.13	1.39	---	---	7311.66	7309.52	---		17177.00
8+63.00	1+44.81	6+46.96	11+50.00	16+66.35	287.00	516.35	216.04	502.15	0.06	1.48	0.08	1.45	---	---	7381.58	7381.12	---		17081.78
9+13.00	1+40.04	6+46.96	11+50.00	16+71.26	237.00	521.26	266.04	506.92	0.07	1.46	0.06	1.47	---	---	7451.67	7451.84	---		16553.47
9+63.00	1+35.23	6+46.96	11+50.00	16+76.20	187.00	526.20	316.04	511.73	0.11	1.42	0.06	1.48	---	---	7521.06	7522.29	---		15994.40
10+13.00	1+30.41	6+46.96	11+50.00	16+81.15	137.00	531.15	366.04	516.55	0.19	1.32	0.05	1.48	---	---	7587.77	7593.07	---		15487.94
10+63.00	1+25.60	6+46.96	11+50.00	16+86.10	87.00	536.10	416.04	521.36	0.37	1.14	0.05	1.47	---	---	7641.91	7663.85	---		15353.15
11+13.00	1+20.78	6+46.96	11+50.00	16+91.06	37.00	541.06	466.04	526.18	0.84	0.65	0.05	1.47	---	---	7604.46	7734.48	---		15356.54
11+63.00	1+15.95	6+46.96	11+50.00	16+96.03	516.04	531.01	13.00	533.03	0.05	1.47	0.25	1.48	3.38	138.56	7805.57	-29.84	7162.89		14946.08
12+13.00	1+11.13	6+46.96	11+50.00	17+00.99	566.04	535.83	63.00	487.99	0.04	1.50	0.98	1.48	16.37	126.86	7877.10	-559.12	6589.89		13910.07
12+63.00	1+06.29	6+46.96	11+50.00	17+05.96	616.04	540.67	113.00	442.96	0.03	1.51	1.25	1.49	29.37	115.15	7948.28	-1282.77	5989.34		12655.71
13+13.00	1+01.54	6+46.96	11+50.00	17+10.85	666.04	545.42	163.00	397.85	0.03	1.51	1.32	1.47	42.37	103.42	8018.11	-1963.36	5314.75		11370.15
13+63.00	0+97.49	6+46.96	11+50.00	17+15.00	716.04	549.47	213.00	352.00	0.02	1.52	1.41	1.47	55.37	91.50	8077.67	-2729.34	4714.71		10063.31
14+13.00	1+22.26	6+46.96	11+50.00	16+89.54	766.04	524.70	263.00	276.54	0.02	1.53	1.45	1.46	68.36	71.89	7713.64	-3474.39	3668.37		7907.75
14+63.00	1+84.82	6+46.96	11+50.00	16+25.20	816.04	462.14	313.00	162.20	0.02	1.52	1.45	1.34	81.36	42.17	6793.99	-4125.56	1974.68		4643.27
15+13.00	2+47.39	6+46.96	11+50.00	15+60.87	866.04	399.57	363.00	47.87	0.02	1.51	1.43	0.75	94.36	12.44	5873.71	-4729.75	328.85		1473.08

XSEC SN 4	Primary Consolidation - Settlement (in)	Differential Settlement (in)	Top of ground post settlement (ft)	Distance between top of ground (ft)		Strain	Pre-settlement slope	Post settlement slope
Cross Section Station	overconsolidated clays	overconsolidated clays	overconsolidated clays	original distance between	final distance between (overconsolidated clays)	overconsolidated clays	Slope at top of native soil	overconsolidated clays
4+63.00	28.94	---	564.10	---	---	---	---	---
5+13.00	31.06	2.11	547.58	52.60	52.66	0.1044%	-32.68%	-33.03%
5+63.00	33.63	2.57	531.00	52.61	52.68	0.1276%	-32.74%	-33.17%
6+13.00	37.09	3.46	514.35	52.61	52.70	0.1716%	-32.72%	-33.30%
6+63.00	49.37	12.28	496.96	52.61	52.94	0.6223%	-32.74%	-34.79%
7+13.00	35.75	13.62	482.31	52.43	52.10	0.6300%	-31.56%	-29.29%
7+63.00	32.31	3.45	481.32	50.02	50.01	0.0130%	-2.56%	-1.99%
8+13.00	33.21	0.90	479.96	50.02	50.02	0.0039%	-2.56%	-2.71%
8+63.00	28.43	4.78	479.09	50.02	50.01	0.0170%	-2.54%	-1.74%
9+13.00	28.49	0.06	477.81	50.02	50.02	0.0003%	-2.56%	-2.57%
9+63.00	30.91	2.42	476.32	50.02	50.02	0.0111%	-2.56%	-2.96%
10+13.00	36.63	5.72	474.56	50.02	50.03	0.0291%	-2.58%	-3.53%
10+63.00	42.65	6.02	472.77	50.02	50.03	0.0309%	-2.58%	-3.58%
11+13.00	51.22	8.57	470.76	50.02	50.04	0.0470%	-2.58%	-4.01%
11+63.00	52.59	1.37	469.36	50.02	50.02	0.0062%	-2.58%	-2.81%
12+13.00	43.15	9.44	468.85	50.02	50.00	0.0282%	-2.58%	-1.01%
12+63.00	36.73	6.42	468.10	50.02	50.01	0.0219%	-2.58%	-1.51%
13+13.00	37.96	1.23	466.73	50.02	50.02	0.0054%	-2.54%	-2.75%
13+63.00	30.43	7.54	466.27	50.01	50.00	0.0192%	-2.16%	-0.90%
14+13.00	24.26	6.17	473.41	50.44	50.51	0.1388%	13.24%	14.27%
14+63.00	24.19	0.07	490.13	52.72	52.72	0.0034%	33.44%	33.45%
15+13.00	25.09	0.90	506.79	52.72	52.70	0.0450%	33.46%	33.31%

XSEC WE 1	Foundation Soil				Groundwater		Vertical stress after excavation	Embankment Fill - Waste		
Cross Section Station	Top of ground elevation (ft)	Approximate top of rock elevation (ft)	Height of Soil Column, H (ft)	Depth from ground to midpoint soil column, z (ft)	Water table elevation (ft)	Pore water pressure, u (psf)	Overburden stress, s'_z (psf)	Top of fill elevation (ft)	Bottom of fill elevation (ft)	Assumed embankment height, H_{emb} (ft)
3+02.63	476.95	366.68	110.27	55.14	452.14	1892.28	4723.92	682.08	476.95	205.13
3+52.63	463.33	373.22	90.11	45.06	453.37	2189.93	3216.67	682.08	463.33	218.75
4+02.63	464.57	378.54	86.03	43.02	454.57	2060.14	3101.66	682.08	464.57	217.51
4+52.63	465.73	381.51	84.22	42.11	455.78	2006.78	3046.42	682.08	465.73	216.35
5+02.63	466.91	383.82	83.09	41.55	456.93	1969.66	3015.74	682.08	466.91	215.17
5+52.63	468.09	388.03	80.06	40.03	458.11	1875.12	2928.48	682.08	468.09	213.99
6+02.63	469.23	390.45	78.78	39.39	459.24	1834.56	2892.24	682.08	469.23	212.85
6+52.63	470.40	393.55	76.85	38.43	460.40	1773.72	2837.28	682.08	470.40	211.68
7+02.63	471.63	396.11	75.52	37.76	461.63	1732.22	2798.98	682.08	471.63	210.45
7+52.63	472.86	397.50	75.36	37.68	462.86	1727.23	2794.37	682.08	472.86	209.22
8+02.63	474.09	399.54	74.55	37.28	464.09	1701.96	2771.04	682.08	474.09	207.99
8+52.63	475.32	404.19	71.13	35.57	465.32	1595.26	2672.54	682.08	475.32	206.76
9+02.63	476.95	410.86	66.09	33.05	466.55	1413.05	2552.35	682.08	476.55	205.53
9+52.63	477.78	417.87	59.91	29.96	467.78	1245.19	2349.41	682.08	477.78	204.30
10+02.63	479.02	422.19	56.83	28.42	469.02	1149.10	2260.70	682.08	479.02	203.06
10+52.63	480.25	417.71	62.54	31.27	470.25	1327.25	2425.15	682.08	480.25	201.83
11+02.63	481.47	414.38	67.09	33.55	471.47	1469.21	2556.19	682.08	481.47	200.61
11+52.63	482.69	406.05	76.64	38.32	472.69	1767.17	2831.23	682.08	482.69	199.39
12+02.63	483.91	414.19	69.72	34.86	473.91	1551.26	2631.94	682.08	483.92	198.16
12+52.63	493.26	433.63	59.63	29.82	475.12	728.52	2849.28	682.08	493.26	188.82
13+02.63	510.40	438.79	71.61	35.81	476.34	108.89	4187.71	682.08	510.40	171.68
13+52.63	527.53	437.05	90.48	45.24	477.56	0.00	5428.80	682.08	527.53	154.55
14+02.63	544.67	427.63	117.04	58.52	478.77	0.00	7022.40	682.08	544.67	137.41
14+52.63	561.81	413.93	147.88	73.94	479.98	0.00	8872.80	682.08	561.81	120.27

Assumed Parameters		
Unit weight of foundation soil, g_s	120.00	pcf
Unit weight of waste, g_{waste}	110.00	pcf
Unit weight of water, g_w	62.40	pcf
Foundation soil in situ void ratio, e_0	0.90	
Foundation soil compression index, C_c	0.20	
Foundation soil swell index, C_r	0.04	
Preconsolidation pressure, s'_c	2600.00	psf

XSEC WE 1	Embankment Load - Vertical stress increase																
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	a_1 (rad)	a_2 (rad)	a_3 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)	Ds' (psf)
3+02.63	3+02.63	10+36.01	10+36.01	17+53.47	733.38	717.46	---	0.04	1.50	---	---	---	11281.38	-10743.19	---	538.19
3+52.63	2+53.96	10+36.01	10+36.01	18+01.09	683.38	765.08	98.67	0.03	1.50	1.14	191.15	27.60	12030.74	-10072.62	1104.05	3062.17
4+02.63	2+58.37	10+36.01	10+36.01	17+96.77	633.38	760.76	144.26	0.04	1.50	1.28	177.16	40.35	11962.53	-9323.17	1809.84	4449.19
4+52.63	2+62.53	10+36.01	10+36.01	17+92.70	583.38	756.69	190.10	0.04	1.50	1.35	163.18	53.17	11898.66	-8563.19	2518.51	5853.98
5+02.63	2+66.73	10+36.01	10+36.01	17+88.59	533.38	752.58	235.90	0.05	1.49	1.40	149.19	65.98	11833.66	-7799.39	3226.17	7260.43
5+52.63	2+70.95	10+36.01	10+36.01	17+84.46	483.38	748.45	281.68	0.05	1.49	1.43	135.21	78.78	11768.68	-7045.39	3943.51	8666.80
6+02.63	2+75.03	10+36.01	10+36.01	17+80.47	433.38	744.46	327.60	0.06	1.48	1.45	121.22	91.63	11705.82	-6282.38	4655.73	10079.16
6+52.63	2+79.21	10+36.01	10+36.01	17+76.38	383.38	740.37	373.42	0.07	1.47	1.47	107.24	104.44	11641.27	-5523.11	5369.23	11487.39
7+02.63	2+83.61	10+36.01	10+36.01	17+72.08	333.38	736.07	419.02	0.08	1.46	1.48	93.25	117.20	11573.30	-4760.50	6077.20	12889.99
7+52.63	2+88.00	10+36.01	10+36.01	17+67.78	283.38	731.77	464.63	0.10	1.44	1.49	79.26	129.96	11505.07	-3992.44	6779.58	14292.21
8+02.63	2+92.40	10+36.01	10+36.01	17+63.48	233.38	727.47	510.23	0.12	1.41	1.50	65.28	142.71	11436.51	-3228.39	7484.65	15692.78
8+52.63	2+96.81	10+36.01	10+36.01	17+59.17	183.38	723.16	555.82	0.15	1.38	1.51	51.29	155.47	11367.62	-2476.93	8203.00	17093.69
9+02.63	3+01.21	10+36.01	10+36.01	17+54.86	133.38	718.85	601.42	0.20	1.33	1.52	37.31	168.22	11297.79	-1734.78	8928.80	18491.80
9+52.63	3+05.62	10+36.01	10+36.01	17+50.55	83.38	714.54	647.01	0.31	1.23	1.52	23.32	180.98	11224.65	-1000.98	9660.73	19884.40
10+02.63	3+10.03	10+36.01	10+36.01	17+46.23	33.38	710.22	692.60	0.67	0.87	1.53	9.34	193.72	11119.64	-283.07	10376.48	21213.05
10+52.63	3+14.43	10+36.01	10+36.01	17+41.92	16.62	721.58	689.29	1.04	0.49	1.53	4.75	197.08	10970.75	-81.25	10526.57	21416.06
11+02.63	3+18.80	10+36.01	10+36.01	17+37.66	66.62	717.21	635.03	0.42	1.10	1.52	19.05	181.56	11009.57	-736.61	9650.30	19923.26
11+52.63	3+23.16	10+36.01	10+36.01	17+33.39	116.62	712.85	580.76	0.27	1.25	1.50	33.34	166.05	10954.02	-1463.09	8749.68	18240.61
12+02.63	3+27.51	10+36.01	10+36.01	17+29.13	166.62	708.50	526.50	0.17	1.36	1.50	47.64	150.52	10894.13	-2276.17	7930.16	16548.11
12+52.63	3+60.94	10+36.01	10+36.01	16+96.43	216.62	675.07	443.80	0.10	1.43	1.50	61.94	126.88	10383.39	-3110.06	6680.39	13953.72
13+02.63	4+22.21	10+36.01	10+36.01	16+36.49	266.62	613.80	333.86	0.09	1.44	1.46	76.23	95.45	9440.51	-3836.34	4892.69	10496.86
13+52.63	4+83.48	10+36.01	10+36.01	15+76.55	316.62	552.53	223.92	0.09	1.43	1.37	90.53	64.02	8497.67	-4529.28	3074.23	7042.62
14+02.63	5+44.75	10+36.01	10+36.01	15+16.62	366.62	491.26	113.99	0.09	1.41	1.10	104.82	32.59	7553.64	-5184.17	1251.23	3620.71
14+52.63	6+06.01	10+36.01	10+36.01	14+56.68	416.62	430.00	4.05	0.09	1.40	0.05	119.12	1.15	6609.22	-5819.00	2.20	792.43

XSEC WE 1	Primary Consolidation - Settlement (in)	Differential Settlement (in)	Top of ground post settlement (ft)	Distance between top of ground (ft)		Strain	Pre-settlement slope	Post settlement slope
Cross Section Station	overconsolidated clays	overconsolidated clays	overconsolidated clays	original distance between	final distance between (overconsolidated clays)	overconsolidated clays	Slope at top of native soil	overconsolidated clays
3+02.63	17.71	---	475.47	---	---	---	---	---
3+52.63	20.74	3.03	461.60	51.82	51.89	0.1291%	-27.24%	-27.74%
4+02.63	24.33	3.59	462.54	50.02	50.01	0.0130%	2.48%	1.88%
4+52.63	27.70	3.37	463.42	50.01	50.01	0.0114%	2.32%	1.76%
5+02.63	30.65	2.95	464.36	50.01	50.01	0.0104%	2.36%	1.87%
5+52.63	32.31	1.66	465.40	50.01	50.01	0.0062%	2.36%	2.08%
6+02.63	34.27	1.96	466.37	50.01	50.01	0.0069%	2.28%	1.95%
6+52.63	35.60	1.33	467.43	50.01	50.01	0.0049%	2.34%	2.12%
7+02.63	36.93	1.32	468.55	50.02	50.01	0.0052%	2.46%	2.24%
7+52.63	38.62	1.69	469.64	50.02	50.01	0.0065%	2.46%	2.18%
8+02.63	39.82	1.20	470.77	50.02	50.01	0.0047%	2.46%	2.26%
8+52.63	39.47	0.36	472.03	50.02	50.02	0.0015%	2.46%	2.52%
9+02.63	37.97	1.49	473.79	50.03	50.03	0.0084%	3.26%	3.51%
9+52.63	35.60	2.37	474.81	50.01	50.01	0.0074%	1.66%	2.06%
10+02.63	34.74	0.86	476.13	50.02	50.02	0.0037%	2.48%	2.62%
10+52.63	38.25	3.52	477.06	50.02	50.01	0.0127%	2.46%	1.87%
11+02.63	39.76	1.51	478.16	50.01	50.01	0.0058%	2.44%	2.19%
11+52.63	43.63	3.87	479.05	50.01	50.01	0.0137%	2.44%	1.79%
12+02.63	38.17	5.46	480.73	50.01	50.03	0.0263%	2.44%	3.35%
12+52.63	30.22	7.95	490.74	50.87	50.99	0.2475%	18.70%	20.02%
13+02.63	32.13	1.91	507.72	52.86	52.80	0.0973%	34.28%	33.96%
13+52.63	35.26	3.12	524.59	52.85	52.77	0.1586%	34.26%	33.74%
14+02.63	38.87	3.61	541.43	52.86	52.76	0.1830%	34.28%	33.68%
14+52.63	43.30	4.43	558.20	52.86	52.74	0.2245%	34.28%	33.54%

XSEC WE 2	Foundation Soil				Groundwater		Vertical stress after excavation	Embankment Fill - Waste		
Cross Section Station	Top of ground elevation (ft)	Approximate top of rock elevation (ft)	Height of Soil Column, H (ft)	Depth from ground to midpoint soil column, z (ft)	Water table elevation (ft)	Pore water pressure, u (psf)	Overburden stress, s'_z (psf)	Top of fill elevation (ft)	Bottom of fill elevation (ft)	Assumed embankment height, H_{emb} (ft)
3+34.00	489.13	350.82	138.31	69.16	455.73	2231.11	6067.49	669.86	489.13	180.73
3+84.00	473.78	353.80	119.98	59.99	457.65	2736.86	4461.94	669.86	473.78	196.08
4+34.00	468.93	358.15	110.78	55.39	458.94	2832.96	3813.84	669.86	468.93	200.93
4+84.00	470.11	362.58	107.53	53.77	460.11	2730.94	3720.86	669.86	470.11	199.75
5+34.00	471.34	366.76	104.58	52.29	461.34	2638.90	3635.90	669.86	471.34	198.52
5+84.00	472.57	369.76	102.81	51.41	462.57	2583.67	3584.93	669.86	472.57	197.29
6+34.00	473.80	372.49	101.31	50.66	463.80	2536.87	3541.73	669.86	473.80	196.06
6+84.00	475.02	375.42	99.60	49.80	465.02	2483.52	3492.48	669.86	475.02	194.84
7+34.00	476.25	380.26	95.99	48.00	466.25	2370.89	3388.51	669.86	476.25	193.61
7+84.00	477.48	379.25	98.23	49.12	467.48	2440.78	3453.02	669.86	477.48	192.38
8+34.00	478.72	388.38	90.34	45.17	468.72	2194.61	3225.79	669.86	478.72	191.14
8+84.00	479.95	401.98	77.97	38.99	469.95	1808.66	2869.54	669.86	479.95	189.91
9+34.00	481.17	411.73	69.44	34.72	471.17	1542.53	2623.87	669.86	481.17	188.69
9+84.00	482.40	417.72	64.68	32.34	472.40	1394.02	2486.78	669.86	482.40	187.46
10+34.00	483.62	435.39	48.23	24.12	473.62	880.78	2013.02	669.86	483.62	186.24
10+84.00	484.84	446.04	38.80	19.40	474.84	586.56	1741.44	669.86	484.84	185.02
11+34.00	486.06	452.29	33.77	16.89	476.06	429.62	1596.58	669.86	486.06	183.80
11+84.00	487.28	456.48	30.80	15.40	477.28	336.96	1511.04	669.86	487.28	182.58
12+34.00	488.50	459.37	29.13	14.57	478.50	284.86	1462.94	669.86	488.50	181.36
12+84.00	503.66	460.16	43.50	21.75	479.71	0.00	2610.00	669.86	503.66	166.20
13+34.00	520.80	457.78	63.02	31.51	480.95	0.00	3781.20	669.86	520.80	149.06
13+84.00	537.94	451.39	86.55	43.28	482.19	0.00	5193.00	669.86	537.94	131.92
14+34.00	555.07	439.93	115.14	57.57	483.45	0.00	6908.40	669.86	555.07	114.79
14+84.00	572.21	427.92	144.29	72.15	484.73	0.00	8657.40	669.86	572.21	97.65

Assumed Parameters		
Unit weight of foundation soil, g_s	120.00	pcf
Unit weight of waste, g_{waste}	110.00	pcf
Unit weight of water, g_w	62.40	pcf
Foundation soil in situ void ratio, e_0	0.90	
Foundation soil compression index, C_c	0.20	
Foundation soil swell index, C_r	0.04	
Preconsolidation pressure, s'_c	2600.00	psf

XSEC WE 2	Embankment Load - Vertical stress increase																	
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	a$_1$ (rad)	a$_2$ (rad)	a$_3$ (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'$_{z1}$ (psf)	Ds'$_{z2}$ (psf)	Ds'$_{z3}$ (psf)	Ds' (psf)
3+34.00	3+34.00	10+89.49	10+89.49	18+48.20	755.49	758.71	755.49	0.05	1.48	1.48	---	---	9938.95	-9362.51	---	576.44
3+84.00	2+69.81	10+89.49	10+89.49	19+12.65	705.49	823.16	114.19	0.05	1.49	1.09	168.77	27.31	10783.46	-8781.07	1039.50	3041.89
4+34.00	2+49.54	10+89.49	10+89.49	19+33.02	655.49	843.53	184.46	0.05	1.49	1.28	156.81	44.12	11050.27	-8161.69	1975.95	4864.53
4+84.00	2+54.49	10+89.49	10+89.49	19+28.04	605.49	838.55	229.51	0.05	1.48	1.34	144.85	54.90	10985.28	-7517.57	2577.16	6044.87
5+34.00	2+59.62	10+89.49	10+89.49	19+22.90	555.49	833.41	274.38	0.06	1.48	1.38	132.88	65.64	10917.52	-6871.72	3177.39	7223.19
5+84.00	2+64.75	10+89.49	10+89.49	19+17.74	505.49	828.25	319.25	0.06	1.47	1.41	120.92	76.37	10849.69	-6221.52	3773.45	8401.63
6+34.00	2+69.88	10+89.49	10+89.49	19+12.59	455.49	823.10	364.12	0.07	1.46	1.43	108.96	87.10	10781.79	-5570.26	4368.94	9580.48
6+84.00	2+75.02	10+89.49	10+89.49	19+07.43	405.49	817.94	408.98	0.08	1.45	1.45	97.00	97.84	10714.36	-4919.96	4966.10	10760.50
7+34.00	2+80.16	10+89.49	10+89.49	19+02.26	355.49	812.77	453.84	0.09	1.44	1.47	85.04	108.57	10646.37	-4277.61	5570.82	11939.58
7+84.00	2+85.30	10+89.49	10+89.49	18+97.10	305.49	807.61	498.70	0.12	1.41	1.47	73.08	119.30	10577.69	-3611.50	6151.43	13117.62
8+34.00	2+90.45	10+89.49	10+89.49	18+91.93	255.49	802.44	543.55	0.13	1.40	1.49	61.12	130.02	10509.08	-2987.11	6773.64	14295.61
8+84.00	2+95.60	10+89.49	10+89.49	18+86.76	205.49	797.27	588.40	0.15	1.38	1.50	49.16	140.75	10441.39	-2381.08	7415.20	15475.52
9+34.00	3+00.71	10+89.49	10+89.49	18+81.62	155.49	792.13	633.29	0.18	1.35	1.52	37.20	151.49	10373.41	-1759.85	8041.43	16654.99
9+84.00	3+05.83	10+89.49	10+89.49	18+76.48	105.49	786.99	678.17	0.26	1.27	1.52	25.23	162.23	10302.41	-1124.86	8651.98	17829.53
10+34.00	3+10.94	10+89.49	10+89.49	18+71.35	55.49	781.86	723.06	0.38	1.16	1.54	13.27	172.97	10231.86	-539.37	9311.44	19003.93
10+84.00	3+16.04	10+89.49	10+89.49	18+66.23	5.49	776.74	767.96	1.27	0.28	1.55	14.27	170.75	10073.63	-137.79	9240.25	19176.08
11+34.00	3+21.14	10+89.49	10+89.49	18+61.11	44.51	768.35	727.11	0.34	1.21	1.55	10.60	173.20	10102.77	-448.43	9385.20	19039.54
11+84.00	3+26.24	10+89.49	10+89.49	18+55.98	94.51	763.25	671.98	0.14	1.41	1.55	22.51	160.07	10040.79	-1110.74	8675.43	17605.48
12+34.00	3+31.33	10+89.49	10+89.49	18+50.88	144.51	758.16	616.88	0.08	1.47	1.55	34.42	146.94	9974.40	-1772.04	7960.25	16162.61
12+84.00	3+94.73	10+89.49	10+89.49	17+87.21	194.51	694.76	503.21	0.09	1.46	1.53	46.33	119.87	9140.28	-2367.51	6411.55	13184.33
13+34.00	4+66.37	10+89.49	10+89.49	17+15.26	244.51	623.12	381.26	0.09	1.44	1.49	58.24	90.82	8196.97	-2941.85	4732.88	9988.00
13+84.00	5+38.00	10+89.49	10+89.49	16+43.32	294.51	551.49	259.32	0.09	1.42	1.41	70.15	61.77	7253.34	-3499.90	3039.72	6793.16
14+34.00	6+09.64	10+89.49	10+89.49	15+71.38	344.51	479.85	137.38	0.10	1.41	1.17	82.06	32.73	6309.82	-4037.55	1345.38	3617.65
14+84.00	6+81.28	10+89.49	10+89.49	14+99.43	394.51	408.21	15.43	0.09	1.39	0.21	93.97	3.68	5365.77	-4573.23	27.15	819.69

XSEC WE 2	Primary Consolidation - Settlement (in)	Differential Settlement (in)	Top of ground post settlement (ft)	Distance between top of ground (ft)		Strain	Pre-settlement slope	Post settlement slope
Cross Section Station	overconsolidated clays	overconsolidated clays	overconsolidated clays	original distance between	final distance between (overconsolidated clays)	overconsolidated clays	Slope at top of native soil	overconsolidated clays
3+34.00	29.16	---	486.70	---	---	---	---	---
3+84.00	31.33	2.16	471.17	52.30	52.36	0.1017%	-30.70%	-31.06%
4+34.00	34.30	2.97	466.07	50.23	50.26	0.0488%	-9.70%	-10.20%
4+84.00	36.92	2.62	467.03	50.01	50.01	0.0094%	2.36%	1.92%
5+34.00	39.08	2.16	468.08	50.02	50.01	0.0082%	2.46%	2.10%
5+84.00	41.29	2.20	469.13	50.02	50.01	0.0084%	2.46%	2.09%
6+34.00	43.27	1.98	470.19	50.02	50.01	0.0076%	2.46%	2.13%
6+84.00	44.87	1.60	471.28	50.01	50.01	0.0062%	2.44%	2.17%
7+34.00	45.32	0.45	472.47	50.02	50.01	0.0018%	2.46%	2.39%
7+84.00	48.37	3.06	473.45	50.02	50.01	0.0112%	2.46%	1.95%
8+34.00	46.21	2.17	474.87	50.02	50.02	0.0096%	2.48%	2.84%
8+84.00	41.36	4.84	476.50	50.02	50.03	0.0231%	2.46%	3.27%
9+34.00	38.13	3.24	477.99	50.01	50.02	0.0146%	2.44%	2.98%
9+84.00	36.63	1.49	479.35	50.02	50.02	0.0064%	2.46%	2.71%
10+34.00	28.32	8.31	481.26	50.01	50.04	0.0433%	2.44%	3.82%
10+84.00	23.04	5.28	482.92	50.01	50.03	0.0253%	2.44%	3.32%
11+34.00	20.09	2.95	484.39	50.01	50.02	0.0132%	2.44%	2.93%
11+84.00	17.77	2.32	485.80	50.01	50.02	0.0102%	2.44%	2.83%
12+34.00	16.21	1.56	487.15	50.01	50.02	0.0067%	2.44%	2.70%
12+84.00	21.52	5.31	501.87	52.25	52.12	0.2423%	30.32%	29.44%
13+34.00	27.52	6.00	518.51	52.86	52.70	0.3028%	34.28%	33.28%
13+84.00	33.00	5.48	535.19	52.86	52.71	0.2766%	34.28%	33.37%
14+34.00	37.99	4.99	551.90	52.85	52.72	0.2524%	34.26%	33.43%
14+84.00	41.67	3.68	568.74	52.86	52.76	0.1865%	34.28%	33.67%

XSEC WE 3	Foundation Soil				Groundwater		Vertical stress after excavation	Embankment Fill - Waste		
Cross Section Station	Top of ground elevation (ft)	Approximate top of rock elevation (ft)	Height of Soil Column, H (ft)	Depth from ground to midpoint soil column, z (ft)	Water table elevation (ft)	Pore water pressure, u (psf)	Overburden stress, s'_z (psf)	Top of fill elevation (ft)	Bottom of fill elevation (ft)	Assumed embankment height, H_{emb} (ft)
6+57.00	467.50	336.12	131.38	65.69	450.20	3019.54	4863.26	602.41	467.50	134.91
7+07.00	461.38	331.47	129.91	64.96	451.37	3428.57	4366.03	602.41	461.38	141.03
7+57.00	462.52	334.54	127.98	63.99	452.52	3368.98	4309.82	602.41	462.52	139.89
8+07.00	463.65	345.55	118.10	59.05	453.65	3060.72	4025.28	602.41	463.65	138.76
8+57.00	464.80	350.29	114.51	57.26	454.80	2948.71	3921.89	602.41	464.80	137.61
9+07.00	465.91	354.14	111.77	55.89	455.91	2863.22	3842.98	602.41	465.91	136.50
9+57.00	467.00	357.82	109.18	54.59	457.00	2782.42	3768.38	602.41	467.00	135.41
10+07.00	468.10	363.20	104.90	52.45	458.10	2648.88	3645.12	602.41	468.10	134.31
10+57.00	469.19	368.51	100.68	50.34	459.19	2517.22	3523.58	602.41	469.19	133.22
11+07.00	470.29	370.06	100.23	50.12	460.29	2503.18	3510.62	602.41	470.29	132.12
11+57.00	471.53	380.31	91.22	45.61	461.53	2222.06	3251.14	602.41	471.53	130.88
12+07.00	472.76	388.14	84.62	42.31	462.76	2016.14	3061.06	602.41	472.76	129.65
12+57.00	474.00	397.05	76.95	38.48	464.00	1776.84	2840.16	602.41	474.00	128.41
13+07.00	475.23	404.21	71.02	35.51	465.23	1591.82	2669.38	602.41	475.23	127.18
13+57.00	476.47	418.45	58.02	29.01	466.47	1186.22	2294.98	602.41	476.47	125.94
14+07.00	481.67	424.38	57.29	28.65	467.71	916.34	2521.06	602.41	481.67	120.74
14+57.00	498.54	423.61	74.93	37.47	468.94	490.78	4005.02	602.41	498.54	103.87
15+07.00	515.67	417.01	98.66	49.33	470.18	239.62	5679.98	602.41	515.67	86.74
15+57.00	532.81	406.41	126.40	63.20	471.39	111.07	7472.93	602.41	532.81	69.60

Assumed Parameters		
Unit weight of foundation soil, g_s	120.00	pcf
Unit weight of waste, g_{waste}	110.00	pcf
Unit weight of water, g_w	62.40	pcf
Foundation soil in situ void ratio, e_0	0.90	
Foundation soil compression index, C_c	0.20	
Foundation soil swell index, C_r	0.04	
Preconsolidation pressure, s'_c	2600.00	psf

XSEC WE 3	Embankment Load - Vertical stress increase																		
Cross Section Station	embankment start station	embankment crest begin station	embankment crest end station	embankment end station	a	b	c	d	a_1 (rad)	a_2 (rad)	a_3 (rad)	a_4 (rad)	h_{above} (ft)	h_{emb2} (ft)	Ds'_{z1} (psf)	Ds'_{z2} (psf)	Ds'_{z3} (psf)	Ds' (psf)
6+57.00	6+57.00	11+10.00	13+41.00	18+71.93	684.00	530.93	453.00	---	0.04	1.48	1.43	---	---	---	7418.83	-6739.80	---	679.04
7+07.00	6+36.45	11+10.00	13+41.00	18+96.02	634.00	555.02	403.00	70.55	0.05	1.47	1.41	0.83	120.02	21.01	7755.21	-5929.54	608.13	2433.81
7+57.00	6+40.29	11+10.00	13+41.00	18+91.52	584.00	550.52	353.00	116.71	0.05	1.46	1.39	1.07	105.13	34.76	7692.29	-5122.04	1301.40	3871.65
8+07.00	6+44.06	11+10.00	13+41.00	18+87.10	534.00	546.10	303.00	162.94	0.06	1.46	1.38	1.22	90.24	48.52	7630.20	-4355.05	2077.93	5353.07
8+57.00	6+47.92	11+10.00	13+41.00	18+82.58	484.00	541.58	253.00	209.08	0.06	1.45	1.35	1.30	75.35	62.26	7566.72	-3557.08	2841.61	6851.26
9+07.00	6+51.67	11+10.00	13+41.00	18+78.18	434.00	537.18	203.00	255.33	0.07	1.44	1.30	1.36	60.46	76.04	7505.33	-2756.60	3608.50	8357.23
9+57.00	6+55.32	11+10.00	13+41.00	18+73.90	384.00	532.90	153.00	301.68	0.08	1.43	1.23	1.39	45.56	89.85	7444.89	-1959.08	4378.56	9864.37
10+07.00	6+59.03	11+10.00	13+41.00	18+69.56	334.00	528.56	103.00	347.97	0.10	1.42	1.10	1.42	30.67	103.64	7383.84	-1181.05	5157.30	11360.09
10+57.00	6+62.66	11+10.00	13+41.00	18+65.30	284.00	524.30	53.00	394.34	0.11	1.40	0.81	1.44	15.78	117.44	7323.07	-448.17	5937.09	12812.00
11+07.00	6+66.38	11+10.00	13+41.00	18+60.94	234.00	519.94	3.00	440.62	0.14	1.36	0.06	1.46	0.89	131.23	7260.62	-1.86	6697.27	13956.03
11+57.00	6+70.52	11+10.00	13+41.00	18+56.09	184.00	515.09	47.00	439.48	0.18	1.33	0.68	0.80	---	---	7190.96	7101.75	---	14292.71
12+07.00	6+74.67	11+10.00	13+41.00	18+51.23	134.00	510.23	97.00	435.33	0.24	1.26	0.33	1.16	---	---	7119.48	7106.51	---	14225.99
12+57.00	6+78.81	11+10.00	13+41.00	18+46.37	84.00	505.37	147.00	431.19	0.36	1.14	0.19	1.31	---	---	7041.72	7054.34	---	14096.06
13+07.00	6+82.96	11+10.00	13+41.00	18+41.51	34.00	500.51	197.00	427.04	0.74	0.76	0.12	1.39	---	---	6923.58	6991.36	---	13914.95
13+57.00	6+87.12	11+10.00	13+41.00	18+36.64	247.00	422.88	16.00	479.64	0.07	1.45	0.50	1.51	4.06	121.88	6925.51	-71.65	6445.60	13299.46
14+07.00	7+04.58	11+10.00	13+41.00	18+16.17	297.00	405.42	66.00	409.17	0.06	1.47	1.16	1.50	16.77	103.97	6639.94	-681.91	5463.91	11421.94
14+57.00	7+61.22	11+10.00	13+41.00	17+49.79	347.00	348.78	116.00	292.79	0.05	1.46	1.26	1.44	29.47	74.40	5711.72	-1298.50	3760.46	8173.69
15+07.00	8+18.76	11+10.00	13+41.00	16+82.35	397.00	291.24	166.00	175.35	0.05	1.45	1.28	1.30	42.18	44.56	4768.95	-1893.29	2022.93	4898.60
15+57.00	8+76.30	11+10.00	13+41.00	16+14.91	447.00	233.70	216.00	57.91	0.05	1.43	1.29	0.74	54.88	14.72	3825.54	-2471.43	382.30	1736.42

XSEC WE 3	Primary Consolidation - Settlement (in)	Differential Settlement (in)	Top of ground post settlement (ft)	Distance between top of ground (ft)		Strain	Pre-settlement slope	Post settlement slope
Cross Section Station	overconsolidated clays	overconsolidated clays	overconsolidated clays	original distance between	final distance between (overconsolidated clays)	overconsolidated clays	Slope at top of native soil	overconsolidated clays
6+57.00	22.76	---	465.60	---	---	---	---	---
7+07.00	30.56	7.80	458.83	50.37	50.46	0.1650%	-12.24%	-13.54%
7+57.00	36.69	6.13	459.46	50.01	50.00	0.0181%	2.28%	1.26%
8+07.00	38.73	2.03	460.42	50.01	50.01	0.0071%	2.26%	1.92%
8+57.00	42.07	3.34	461.29	50.01	50.01	0.0113%	2.30%	1.74%
9+07.00	45.00	2.93	462.16	50.01	50.01	0.0096%	2.22%	1.73%
9+57.00	47.40	2.40	463.05	50.01	50.01	0.0079%	2.18%	1.78%
10+07.00	48.49	1.09	464.06	50.01	50.01	0.0038%	2.20%	2.02%
10+57.00	49.07	0.58	465.10	50.01	50.01	0.0021%	2.18%	2.08%
11+07.00	50.72	1.64	466.06	50.01	50.01	0.0056%	2.20%	1.93%
11+57.00	46.65	4.06	467.64	50.02	50.02	0.0191%	2.48%	3.16%
12+07.00	43.21	3.44	469.16	50.02	50.02	0.0157%	2.46%	3.03%
12+57.00	39.18	4.03	470.74	50.02	50.02	0.0189%	2.48%	3.15%
13+07.00	35.99	3.19	472.23	50.02	50.02	0.0145%	2.46%	2.99%
13+57.00	28.91	7.09	474.06	50.02	50.03	0.0362%	2.48%	3.66%
14+07.00	26.49	2.42	479.46	50.27	50.29	0.0423%	10.40%	10.80%
14+57.00	29.96	3.47	496.04	52.77	52.68	0.1740%	33.74%	33.16%
15+07.00	33.75	3.79	512.86	52.85	52.75	0.1919%	34.26%	33.63%
15+57.00	36.53	2.78	529.77	52.86	52.78	0.1413%	34.28%	33.82%

Attachment E Settlement Example Calculation

Settlement Analysis

Maximum Settlement Analysis for Scepter Class II Landfill at Scepter Waverly, TN

Prepared for

Scepter, Inc.
1485 Scepter Lane
Waverly, Tennessee 37130

Prepared by

AECOM

1000 Corporate Centre Drive, Suite 250
Franklin, Tennessee 37067-6209

Discussion:

The following calculation is an example of the settlement analysis computed in Microsoft Excel. The settlement potential of the foundations soils is calculated for Scepter, Inc.'s Landfill in Waverly, Tennessee. The calculations are based on the following reference:

1) Das, Braja M. Principles of Geotechnical Engineering, 7th Edition. Stamford: Cengage Learning 2010.

The maximum settlement was found to occur at Cross Section SN3, Station 7+50. The following is a detail calculation for this cross section.

The slope from this specific location (pre-settlement and post-settlement) to the leachate sump are also evaluated below.

Geometry and Soil Parameters:

Unit weight of foundation soil, $\gamma_s := 120\text{pcf}$ Foundation soil in-situ void ratio, $e_0 := 0.90$

Foundation soil compression index, $C_c := 0.20$ Foundation soil swell index, $C_r := 0.04$

Preconsolidation pressure, $\sigma'_c := 2600\text{psf}$

Unit weight of waste, $\gamma_{waste} := 110\text{pcf}$

Unit weight of water, $\gamma_w := 62.4\text{pcf}$

Cross Section Station	Top of Ground Elev.	Approximate Top of Rock Elev.	Water table Elev.
SN3, 7+50	$EL_g := 479.44\text{ft}$	$EL_{tor} := 366.22\text{ft}$	$EL_w := 469.44\text{ft}$

Height of soil column, $H_{sc} := EL_g - EL_{tor} = 113.22\,\text{ft}$

Depth from ground to midpoint soil column, $z := \dfrac{H_{sc}}{2} = 56.61\,\text{ft}$

Pore water pressure, $u := \gamma_w \cdot \left(EL_w - EL_{tor} - z\right) = 2908.46 \cdot \text{psf}$

Vertical stress after excavation, $\sigma'_z := \gamma_s \cdot \left(EL_g - EL_w\right) + \gamma_s \cdot \left(EL_w - EL_{tor} - z\right) - u = 3884.74 \cdot \text{psf}$

Embankment Fill - Waste

Cross Section Station	Top of Fill Elev.	Grade Break Elev.	Bottom of Fill Elev.
SN3, 7+50	$EL_f := 613.71ft$	$EL_{br} := 562.29ft$	$EL_b := 479.44ft$

Assumed upper embankment height, $H_{emba} := EL_f - EL_{br} = 51.42\,ft$

Assumed lower embankment height, $H_{embb} := EL_{br} - EL_b = 82.85\,ft$

Upper embankment load - vertical stress increase (measured in AutoCAD)

$au := 74.22ft$ $\quad bu := 183.79ft$ $\quad cu := 3.18ft$ $\quad du := 307.27ft$

$\alpha u_1 := \operatorname{atan}\left(\frac{au + bu}{z}\right) - \operatorname{atan}\left(\frac{au}{z}\right) = 0.44$ $\qquad \alpha u_2 := \operatorname{atan}\left(\frac{au}{z}\right) = 0.92$

$\alpha u_3 := \operatorname{atan}\left(\frac{cu}{z}\right) = 0.06$ $\qquad \alpha u_4 := \operatorname{atan}\left(\frac{du}{z}\right) = 1.39$

height above upper embankment (airspace), $h_{upab} := 0.53ft$

height of embankment at cross section station, $h_{emb} := 50.89ft$

upper embankment vertical stress increase components,

$$\Delta\sigma'_{z1} := \frac{\gamma_{waste} \cdot H_{emba}}{\pi} \cdot \left[\left(\frac{au + bu}{bu}\right) \cdot (\alpha u_1 + \alpha u_2) - \frac{au}{bu} \cdot \alpha u_2\right] = 2755.95 \cdot psf$$

$$\Delta\sigma'_{z2} := -\frac{\gamma_{waste} \cdot h_{upab}}{\pi} \cdot \alpha u_3 = -1.04 \cdot psf$$

$$\Delta\sigma'_{z3} := \frac{\gamma_{waste} \cdot h_{emb}}{\pi} \cdot \alpha u_4 = 2474.31 \cdot psf$$

Lower embankment load - vertical stress increase (measured in AutoCAD)

$al := 555.87ft$ $bl := 310.23ft$ $cl := 258.01ft$ $dl := 296.11ft$

$$\alpha l_1 := \operatorname{atan}\left(\frac{al + bl}{z}\right) - \operatorname{atan}\left(\frac{al}{z}\right) = 0.04 \qquad \alpha l_2 := \operatorname{atan}\left(\frac{al}{z}\right) = 1.47$$

$$\alpha l_3 := \operatorname{atan}\left(\frac{cl + dl}{z}\right) - \operatorname{atan}\left(\frac{cl}{z}\right) = 0.11 \qquad \alpha l_4 := \operatorname{atan}\left(\frac{cl}{z}\right) = 1.35$$

lower embankment vertical stress increase components,

$$\Delta\sigma'_{z4} := \frac{\gamma_{waste} \cdot H_{embb}}{\pi} \cdot \left[\left(\frac{al + bl}{bl}\right)\cdot\left(\alpha l_1 + \alpha l_2\right) - \frac{al}{bl}\cdot\alpha l_2\right] = 4555.68\cdot psf$$

$$\Delta\sigma'_{z5} := \frac{\gamma_{waste} \cdot H_{embb}}{\pi} \cdot \left[\left(\frac{cl + dl}{dl}\right)\cdot\left(\alpha l_3 + \alpha l_4\right) - \frac{cl}{dl}\cdot\alpha l_4\right] = 4550.02\cdot psf$$

Total vertical stress increase, $\Delta\sigma' := \Delta\sigma'_{z1} + \Delta\sigma'_{z2} + \Delta\sigma'_{z3} + \Delta\sigma'_{z4} + \Delta\sigma'_{z5} = 14334.92\cdot psf$

Calculate settlement of foundation soil due to addition of waste - Assumes overconsolidated clays

$$S_c := \begin{cases} \frac{C_r \cdot z}{1 + e_0}\cdot\log\left(\frac{\sigma'_z + \Delta\sigma'}{\sigma'_z}\right) & \text{if } \sigma'_z + \Delta\sigma' \le \sigma'_c \\ \frac{C_r \cdot z}{1 + e_0}\cdot\log\left(\frac{\sigma'_c}{\sigma'_z}\right) + \frac{C_c \cdot z}{1 + e_0}\cdot\log\left(\frac{\sigma'_z + \Delta\sigma'}{\sigma'_c}\right) & \text{if } \sigma'_z + \Delta\sigma' > \sigma'_c \end{cases} = 57.97\cdot in$$

$S_c = 4.83\,ft$

Calculate ground slope between maximum settlement spot to leachate sump

Distance between points measured in AutoCAD, $dist := 670.89ft$

Top of proposed ground at SN3, Station 7+50, $EL_{pg_1} := 479.44ft$

Top of proposed ground at sump $EL_{pg_2} := 461ft$

Ground elevation post-settlement at SN3, Station 7+50, $EL_{ps_1} := 474.61ft$

It is conservatively assumed that no settlement has occurred at the leachate sump.

Slope of proposed ground, $slope_1 := \frac{EL_{pg_1} - EL_{pg_2}}{dist} = 2.75\cdot\%$

Slope of ground post-settlement, $slope_2 := \frac{EL_{ps_1} - EL_{pg_2}}{dist} = 2.03\cdot\%$

Both the proposed and post-settlement ground slopes are calculated to be greater than 2%.